MIL-HDBK-1021/4
30 APRIL 1987
SUPERSEDING
NAVFAC DM-21
June 1973 (in part)

MILITARY HANDBOOK

RIGID PAVEMENT DESIGN FOR AIRFIELDS

NO DELIVERABLE DATA REQUIRED BY THIS DOCUMENT

AREA FACR

ABSTRACT

Basic criteria regarding rigid airfield pavement design are provided for use by experienced engineers. The contents include criteria on subgrade, stabilized materials, base course, concrete materials, airfield traffic, rigid pavement thickness design, joint design and strengthening of rigid pavements.

FOREWORD

This military handbook is one of a series developed from an evaluation of facilities in the shore establishment, from surveys of the availability of new materials and construction methods, and from selection of the best design practices of the Naval Facilities Engineering Command, other Government agencies, and the private sector. It uses, to the maximum extent feasible, national professional society, association, and institute standards in accordance with NAVFACENGCOM policy. Deviations from these criteria in the planning, engineering, design, and construction of naval shore facilities, cannot be made without prior approval of NAVFACENGCOM Headquarters (Code 04).

Design cannot remain static any more than can the naval functions it serves or the technologies it uses. Accordingly, recommendations for improvement are encouraged from within the Navy and from the private sector and should be furnished to Commander, Western Division, Naval Facilities Engineering Command (Code 406), P.O. Box 727, San Bruno, CA 94066.

THIS HANDBOOK SHALL NOT BE USED AS A REFERENCE DOCUMENT FOR PROCUREMENT OF FACILITIES CONSTRUCTION. IT IS TO BE USED IN THE PURCHASE OF FACILITIES ENGINEERING STUDIES AND DESIGN (FINAL PLANS, SPECIFICATIONS, AND COST ESTIMATES). DO NOT REFERENCE IT IN MILITARY OR FEDERAL SPECIFICATIONS OR OTHER PROCUREMENT DOCUMENTS.

AIRFIELD PAVEMENT CRITERIA MANUALS

Criteria Manual	Title	PA
MIL-HDBK-1021/1	Airfield Geometric Design	SOUTHDIV
MIL-HDBK-1021/2	General Concepts for Pavement Design	WESTDIV
DM-21.03	Flexible Pavement Design for Airfields	ARMY
MIL-HDBK-1021/4	Rigid Pavement Design for Airfields	WESTDIV
DM-21.06	Airfield and Subsurface Drainage Pavement Design for Frost Conditions	SOUTHDIV
DM-21.09	Skid-Resistant Runway Surfaces	HEADQUARTERS

MILITARY HANDBOOK

RIGID PAVEMENT DESIGN FOR AIRFIELDS

TO ALL HOLDERS OF MIL-HDBK-1021/4

1. THE FOLLOWING PAGES OF MIL-HDBK 1021/4 HAVE BEEN REVISED AND SUPERSEDE THE PAGES LISTED:

NEW PAGE	DATE	SUPERSEDED PAGE	DATE
48	July 1988	48	April 1987
49	July 1988	49	April 1987
69	July 1988	69	April 1987
70	July 1988	70	April 1987
71	July 1988	71	April 1987
72	July 1988	72	April 1987
114	July 1988	114	April 1987
115	July 1988	115	April 1987
117	July 1988	117	April 1987
118	July 1988	118	April 1987

2. RETAIN THIS NOTICE AND INSERT BEFORE TABLE OF CONTENTS.

3. Holders of MIL-HDBK-1021/4 will verify that all changes indicated above have been made. This notice page will be retained as a check sheet. This issuance, together with appended pages, is a separate publication. Each notice is to be retained by stocking points until the Military Handbook is completely revised or cancelled.

CUSTODIANS:
NAVY-YD

PREPARING ACTIVITY:
NAVY-YD
PROJECT NO. FACR-0239

DISTRIBUTION STATEMENT A. Approved for public release; distribution is unlimited.

MILITARY HANDBOOK

RIGID PAVEMENT DESIGN FOR AIRFIELDS

TO ALL HOLDERS OF MIL-HDBK-1021/4

1. THE FOLLOWING PAGES OF MIL-HDBK-1021/4 HAVE BEEN REVISED AND SUPERSEDE THE PAGES LISTED:

NEW PAGE	DATE	SUPERSEDED PAGE	DATE
iv	31 August 1992	iv	30 April 1987
v	31 August 1992	v	30 April 1987
ix	31 August 1992	ix	30 April 1987
x	31 August 1992	x	30 April 1987
xi	31 August 1992	xi	30 April 1987
xii	31 August 1992	xii	30 April 1987
xiii	31 August 1992	xiii	30 April 1987
xiv	31 August 1992	xiv	30 April 1987
xv	31 August 1992	New Page	
1	31 August 1992	1	30 April 1987
2	31 August 1992	2	30 April 1987
31	31 August 1992	31	30 April 1987
32	31 August 1992	32	30 April 1987
33	31 August 1992	33	30 April 1987
34	31 August 1992	34	30 April 1987
35	31 August 1992	35	30 April 1987
36	31 August 1992	36	30 April 1987
37	31 August 1992	37	30 April 1987
38	31 August 1992	38	30 April 1987
39	31 August 1992	39	30 April 1987
40	31 August 1992	40	30 April 1987
43	31 August 1992	43	30 April 1987
44	31 August 1992	44	30 April 1987
53	31 August 1992	53	30 April 1987
54	31 August 1992	54	30 April 1987
54a	31 August 1992	New Page	
73	31 August 1992	73	30 April 1987
74	31 August 1992	74	30 April 1987

NEW PAGE	DATE	SUPERSEDED PAGE	DATE
79	31 August 1992	79	30 April 1987
80	31 August 1992	80	30 April 1987
87	31 August 1992	87	30 April 1987
88	31 August 1992	88	30 April 1987
107	31 August 1992	107	30 April 1987
108	31 August 1992	108	30 April 1987
119	31 August 1992	119	30 April 1987
120	31 August 1992	New Page	
121	31 August 1992	121	30 April 1987
122	31 August 1992	122	30 April 1987
123	31 August 1992	123	30 April 1987
124	31 August 1992	124	30 April 1987
125	31 August 1992	125	30 April 1987
126	31 August 1992	126	30 April 1987
127	31 August 1992	New Page	
128	31 August 1992	New Page	
129	31 August 1992	New Page	
130	31 August 1992	New Page	
131	31 August 1992	New Page	
132	31 August 1992	New Page	

2. RETAIN THIS NOTICE AND INSERT BEFORE TABLE OF CONTENTS.

3. Holders of MIL-HDBK-1021/4 will verify that all changes indicated above have been made. This notice page will be retained as a check sheet. This issuance, together with appended pages, is a separate publication. Each notice is to be retained by stocking points until the Military Handbook is completely revised or cancelled.

CUSTODIANS:

NAVY - YD

PREPARING ACTIVITY:
NAVY - YD

PROJECT NO.
FACR-1096

DISTRIBUTION STATEMENT A. Approved for public release; distribution is unlimited.

RIGID PAVEMENT DESIGN FOR AIRFIELDS

CONTENTS

		Page
SECTION 1:	INTRODUCTION	1
1.1	Scope	1
1.2.	Cancellation	1
1.3.	Related Criteria	1
SECTION 2:	DESIGN INPUT DATA	3
2.1	General	3
2.2	Subgrade Soils	3
2.3	Base Course Materials	3
2.4	Concrete Materials	3
2.5	Climate and Drainage Considerations	3
2.6	Traffic Data	3
2.7	Existing Pavement Condition	3
SECTION 3:	SUBGRADE EVALUATION AND PREPARATION	5
3.1	General	5
3.2	Soil Investigation	5
3.2.1	Soil Survey and Sampling	5
3.2.2	Soil Tests	5
3.3	Soil Classification System	5
3.4	Soil Strength Tests	7
3.4.1	Determination of Modulus of Subgrade Reaction (k Value)	7
3.4.2	Number of Tests	10
3.4.3	Correlations With Other Tests	10
3.5	Drainage	10
3.6	Special Problems	10
3.6.1	Frost Heave	10
3.6.2	Swelling Soil	10
3.6.2.1	Moisture Content	12
3.6.2.2	Degree of Compaction	12
3.6.2.3	Plasticity	12
3.6.2.4	Construction Considerations	12
3.6.3	Variability Through Project Area	12
3.7	Stabilization	12
3.8	Imported Fill	12
3.9	Density and Compaction	12
SECTION 4:	SOIL STABILIZATION	15
4.1	Scope	15
4.2	Objectives	15
4.2.1	Construction Expediency	15
4.2.2	Subgrade Modification	15
4.2.3	Strength and Durability Improvements	15
4.3	Types of Stabilizers	15
4.4	Criteria for Requiring Soil Stabilization	15

		Page
4.5	Lime Stabilization	16
4.5.1	Stabilization Mechanisms	16
4.5.1.1	Cation Exchange and Flocculation and Agglomeration	16
4.5.1.2	Soil-Lime Pozzolanic Reaction	16
4.5.2	Suitable Soils for Lime Stabilization	16
4.5.3	Lime Stabilization Construction Sequence	16
4.5.4	Types of Lime Application	16
4.5.4.1	Dry Hydrated Lime	17
4.5.4.2	Quicklime	17
4.5.4.3	Slurry Lime	17
4.5.5	Quality Control and Laboratory Tests	17
4.6	Portland Cement Stabilization	17
4.6.1	Stabilization Mechanisms	17
4.6.2	Suitable Soils for Portland Cement Stabilization	17
4.6.3	Portland Cement Stabilization Construction Sequence	18
4.6.4	Quality Control and Laboratory Tests	18
4.7	Asphalt Stabilization	18
4.7.1	Stabilization Mechanisms	18
4.7.2	Suitable Materials for Asphalt Stabilization	18
4.7.3	Construction Sequence	18
4.7.4	Quality Control and Laboratory Tests	19
4.8	Lime-Fly Ash Stabilization	19
4.9	Thickness of Stabilized Subgrade Layers	19
SECTION 5:	BASE COURSE MATERIALS	21
5.1	Introduction	21
5.2	Materials	21
5.2.1	Requirements for Granular Materials	21
5.2.1.1	Gradation	21
5.2.1.2	Wear Resistance	21
5.2.1.3	Freeze-Thaw Durability	22
5.2.1.4	CBR Requirements	22
5.2.2	Stabilized Bases	22
5.2.3	Lean Concrete Bases	22
5.2.4	Recycled Concrete Bases	22
5.2.5	Geotextile Fabrics	22
5.3	Design Considerations	23
5.3.1	Minimum Thickness Requirements	23
5.3.2	Effective k Value	24
5.3.3	Base Type	24
5.3.4	Gradation	27
5.3.5	Drainage and Permeability	28
5.3.5.1	Permeability	28
5.3.5.2	Filter Layer	28
5.3.6	Drainage Layer Beneath Stabilized Base	28
5.4	Quality Control	28
5.4.1	Compaction	29
5.4.2	Construction Procedure	29
5.4.3	Testing Considerations	29

		Page
SECTION 6:	PORTLAND CEMENT CONCRETE MATERIALS.............	31
6.1	Introduction..................................	31
6.2	Heat Resistant Concrete for F/A-18 Aircraft....	31
6.3	Cement Types..................................	31
6.3.1	Type I..	31
6.3.2	Type II.......................................	31
6.3.3	Type III......................................	31
6.3.4	Type IV.......................................	31
6.3.5	Type V..	31
6.3.6	Criteria for Selection of Cement Types.........	32
6.3.6.1	Chemical Actions..............................	32
6.3.6.2	Construction Considerations...................	32
6.3.6.3	Availability..................................	32
6.4	Concrete Aggregates...........................	33
6.4.1	Aggregate Types...............................	33
6.4.2	Gradation.....................................	33
6.4.3	Maximum Aggregate Size........................	34
6.4.4	Performance of Local Aggregates................	34
6.4.4.1	"D" Cracking..................................	34
6.4.4.2	Reactive Aggregates...........................	34
6.4.5	Recycled Portland Cement Concrete..............	34
6.5	Admixtures....................................	35
6.5.1	Air-Entraining Admixtures.....................	36
6.5.2	Water-Reducing Admixtures.....................	36
6.5.3	Set-Retarding Admixtures......................	36
6.5.4	Set-Accelerating Admixtures...................	36
6.5.5	Mineral Admixtures............................	36
6.5.5.1	Materials of Low Reactivity...................	36
6.5.5.2	Cementitious Materials........................	37
6.5.5.3	Pozzolanic Materials..........................	37
6.5.6	Factors in Selecting Admixtures................	37
6.6	Testing Procedures............................	37
6.6.1	Aggregate Tests...............................	37
6.6.1.1	Sieve Analysis and Gradation..................	37
6.6.1.2	Specific Gravity and Absorption................	37
6.6.1.3	Unit Weight...................................	37
6.6.1.4	Surface Moisture..............................	38
6.6.1.5	Soundness.....................................	38
6.6.1.6	Resistance to Abrasion........................	38
6.6.1.7	Cleanliness and Deleterious Substances.........	38
6.6.2	Concrete Tests................................	38
6.6.2.1	Fresh Concrete Tests..........................	38
6.6.2.2	Hardened Concrete Tests.......................	39
6.7	Mix Design Process............................	40
6.7.1	Mix Design Procedures.........................	40
6.7.2	Design of Lean Concrete Mixtures...............	40
6.8	Hot Weather Considerations.....................	40

		Page
6.8.1	Effects of High Temperatures....................	40
6.8.2	Cooling Concrete Materials......................	41
6.8.3	Transporting, Placing, and Finishing...........	42
6.8.4	Plastic Cracking................................	42
6.8.5	Curing..	43
6.9	Cold Weather Consideratons......................	43
6.9.1	Effects of Cold Weather.........................	43
6.9.2	Heating of Concrete Materials...................	43
6.9.3	Transporting, Placing, and Finishing...........	43
6.9.4	Curing..	43
SECTION 7:	AIRCRAFT TRAFFIC................................	45
7.1	General...	45
7.2	Aircraft Types..................................	45
7.3	Design Weight...................................	46
7.3.1	Use of Other Gear Loads in Design..............	46
7.3.2	Hangar Floors...................................	46
7.4	Traffic Areas...................................	46
7.4.1	Primary Traffic Areas...........................	46
7.4.2	Secondard Traffic Areas.........................	46
7.4.3	Supporting Areas................................	49
7.5	Pass-Coverage Ratio.............................	49
SECTION 8:	THICKNESS DESIGN FOR NON-REINFORCED CONCRETE PAVEMENT..................................	51
8.1	Basis for Design................................	51
8.1.1	Fatigue Damage..................................	51
8.1.2	Structural Characterization.....................	51
8.1.3	Structural Slab Cracking from Aircraft Loadings	51
8.1.4	Structural Slab Cracking annd Mixed Aircraft Loadings..	51
8.2	Thickness Design Inputs.........................	53
8.2.1	Design Concrete Flexural Strength..............	53
8.2.2	k Value at Top of Base..........................	53
8.2.3	Type and Design Gear Load of aircraft Using Facility..................................	53
8.2.4	Number of Aircraft Passes.......................	53
8.2.5	Primary or Secondary Traffic Area..............	54
8.3	Thickness Design Procedures for a Single Design Aircraft..	54
8.4	Thickness Design Procedure for Mixed Traffic...	60
8.4.1	Required Inputs.................................	60
8.4.2	Determination of Interior Flexural Stresses....	60
8.4.3	Fatigue Life Consumption........................	62
8.5	Minimum Thickness...............................	62
8.6	Detailed Design Example.........................	68
8.6.1	Background Information..........................	68

Page

8.6.2	Subgrade Evaluation and Testing	68
8.6.3	Base Course Design	69
8.6.4	Traffic Projections	69
8.6.5	Slab Thickness and Joints	69
SECTION 9:	JOINT DESIGN FOR NON-REINFORCED CONCRETE PAVEMENT	73
9.1	Introduction	73
9.2	Basis for Design	73
9.3	Types of Joints and Uses	73
9.3.1	Expansion Joints	73
9.3.2	Contraction (Weakened Plane) Joints	73
9.3.3	Construction Joints	73
9.3.3.1	Transverse Construction Joints	76
9.3.3.2	Longitudinal Construction Joints	76
9.4	Joint Spacing	76
9.5	Load Transfer Design	76
9.5.1	Aggregate Interlock	76
9.5.2	Dowel Bars	79
9.5.3	Keyways	79
9.5.4	Stabilized Base	79
9.6	Joint Sealants	79
9.6.1	Types of Sealant Materials	79
9.6.1.1	Field Poured, Hot Applied	79
9.6.1.2	Field Poured, Cold Applied	80
9.6.1.3	Preformed Compression Seals	80
9.6.2	Joint Reservoir Design	80
9.6.2.1	Field Poured Sealants	80
9.6.2.2	Preformed Compression Seals	80
9.7	Tiedown Mooring Eyes	80
SECTION 10:	DESIGN OF REINFORCED PAVEMENTS	83
10.1	Introduction	83
10.2	Basis of Design	83
10.2.1	Jointed Reinforced Concrete Pavement	83
10.2.1.1	Thickness	83
10.2.1.2	Reinforcement	83
10.2.1.3	Joints	83
10.2.2	Continuously Reinforced Concrete Pavement	84
10.2.2.1	Thickness	84
10.2.2.2	Reinforcement	84
10.2.2.3	Joints	84
10.2.3	Steel Fiber Reinforced Concrete Pavement	85
10.2.4	Prestressed Concrete Pavement	85
10.2.4.1	Thickness	85
10.2.4.2	Reinforcement	85
10.2.4.3	Joints	85

Page

SECTION 11:	STRENGTHENING OF RIGID PAVEMENTS	87
11.1	General	87
11.2	Rehabilitation Alternative Selection	87
11.2.1	Pavement Evaluation Procedure	87
11.2.1.1	Condition Survey	88
11.2.1.2	Distress Evaluation	90
11.2.1.3	Load-Bearing Capacity	90
11.2.1.4	Functional Condition	90
11.2.1.5	Maintenance History	94
11.2.2	Need for Structural Improvement	94
11.3	Data Collection for Overlay Design	94
11.3.1	Existing Slab Thickness	95
11.3.2	Existing Concrete Flexural Strength	95
11.3.3	Concrete Overlay Flexural Strength	95
11.3.4	Effective k Value	95
11.3.5	Load Transfer Measurement	96
11.4	Overlay Types	97
11.4.1	Unbonded Concrete Overlay	97
11.4.2	Bonded Concrete Overlay	97
11.4.3	Partially Bonded Concrete Overlay	97
11.4.4	Asphalt Concrete Overlay	97
11.4.5	Reinforced Concrete Overlays	98
11.4.5.1	Jointed Reinforced	98
11.4.5.2	Continuously Reinforced	98
11.4.5.3	Steel Fiber Reinforced	98
11.4.6	Selection of Overlay Type	99
11.5	Pre-Overlay Repair	100
11.5.1	Localized Repair	100
11.5.2	Bonded and Partially Bonded Concrete Overlays	100
11.5.3	Unbonded Concrete Overlay	100
11.5.4	Asphalt Concrete Overlays	102
11.6	Preparation of Existing Surface	102
11.6.1	Bonded Concrete Overlay	102
11.6.2	Partially Bonded Concrete Overlay	102
11.6.3	Unbonded Concrete Overlay	102
11.6.4	Asphalt Concrete Overlay	103
11.7	Overlay Thickness Design	104
11.7.1	Concrete Overlay Thickness Design	104
11.7.1.1	Required Inputs	104
11.7.1.2	Determination of Flexural Stress	104
11.7.1.3	Fatigue Life Consumption	107
11.7.1.4	Determination of Cr	107
11.7.1.5	Determination of Overlay Thickness (To)	108
11.7.2	Asphalt Concrete Overlay Thickness Design	108
11.7.2.1	Determination of Cb	108
11.7.2.2	Determination of F	108

Page

11.7.2.3 Determination of Asphalt Concrete Overlay Thickness.. 110
11.8 Joint Design...................................... 110
11.8.1 Bonded and Partially Bonded Concrete Overlays...... 110
11.8.2 Unbonded Concrete Overlay.......................... 110
11.8.3 Asphalt Concrete Overlay........................... 110
11.9 Overlay Design Examples............................ 110
11.9.1 Concrete Overlay Design Example.................... 113
11.9.2 Asphalt Concrete Overlay Design Example............ 119

SECTION 12: AIRCRAFT AND ENGINE RESTRAINT SYSTEM PROOF LOAD TESTING REQUIREMENTS.................. 120
12.1 Purpose.. 120
12.2 Inspection and Test Frequency...................... 120
12.3 Description of System.............................. 120
12.4 Proof Loads.. 120
12.5 Inspection and Test Procedures..................... 122
12.6 Scheduling and Performance Responsibility.......... 123
12.7 Inspection and Test Results........................ 123
12.7.1 Inspections and Tests by Activity.................. 123
12.7.2 Inspections and Tests by Engineering Field Division.. 123
12.7.3 Action and Deficiencies............................ 123
12.8 Forms.. 124

FIGURES

1 Relationship of Soil Classifications and Bearing Values to k Values.................................... 11
2 Determination of Effective k Value on Top of Granular Base Course............................... 26
3 Multiwheel Landing Gear Assemblies........................ 47
4 Rigid Pavement Thickness Design Chart for Single Wheel Gear.. 55
5 Rigid Pavement Thickness Design Chart for P-3 Aircraft.. 56
6 Rigid Pavement Thickness Design Chart for C-130 Aircraft.. 57
7 Rigid Pavement Thickness Design Chart for C-141 Aircraft.. 58
8 Rigid Pavement Thickness Design Chart for C-5A Aircraft.. 59
9 Chart for Determining Flexural Stress for Single Wheel Gear.. 63
10 Chart for Determining Flexural Stress for P-3 Aircraft.. 64

Page

FIGURES (Cont.)

11	Chart for Determining Flexural Stress for C-130 Aircraft	65
12	Chart for Determining Flexural Stress for C-141 Aircraft	66
13	Chart for Determining Flexural Stress for C-5A Aircraft	67
14	Expansion Joint Details	74
15	Contraction (Weakened Plane) Joint Details	75
16	Construction Joint Details	77
17	Typical Longitudinal and Transverse Joint Layout	78
18	Tiedown Mooring Eye Details	81
19	PCI Scale and Condition Rating	89
20	Example PCI Profile for a Runway Center and Edges	91
21	Example Distribution of Distressed Slabs Across Runway	92
22	Example Plot of PCI Over Time for a Specific Pavement	93
23	General Guide for Type of Overlay Selection	101
24	Chart for Determining Load Transfer Adjustment Factor for Various Aircraft	106
25	Chart for Determining Cr (for Rigid Overlays)	109
26	Chart for Determining Cb (for Flexible Overlays)	111
27	Chart for Determining F Factor for Design of Flexible Pavement	112
28	Proof Load Test System	121

TABLES

1	Minimum Requirements for Spacing and Depth of Exploratory Borings	6
2	Subgrade Sampling and Testing Standards	7
3	Subgrade Testing Standards	7
4	Soil Characteristics Pertinent to Roads and Airfields	8
5	Gradations for Lean Concrete Base Materials	23
6	Base Course Minimum Thickness Requirements	23
7	Test Methods for Base Course Materials	29
8	Types of Cement Required for Concrete Exposed to Sulfate Attack	32
9	Admixture Effects on Concrete Properties	35
10	Tests for Deleterious Substances	38
11	Fresh Concrete Tests	39
12	Hardened Concrete Tests	39

Page

TABLES

13 Standard Design Aircraft Types.................................. 45
14 Commonly Used Design Loadings for Navy and Marine
Corps Air Stations.. 48
15 Pass-Coverage Ratios for Standard Aircraft..................... 49
16 Minimum Pass Levels for Standard Aircraft...................... 50
17 Stress-Strength Ratios and Allowable Coverages................. 52
18 Design Example for a Single Design Aircraft.................... 54a
19 Fatigue Damage Summary Sheet for Mixed Traffic................. 61
20 Design Example for Primary (Channelized) Traffic
Areas... 70
21 Design Example for Secondary (Unchannelized)
Traffic Areas... 71
22 Fatigue Damage Summary Sheet for Overlay Design................ 105
23 Design Example for Single Slab Thickness for Bonded
Bonded or Partially Bonded Concrete Overlay................. 114
24 Design Example for Single Slab Thickness for
Unbonded Concrete Overlay................................... 117
25 Jet Engine Test Cells and Unabated Power Check
Fac. Restraint System Proof Load/Pull Angles................ 124
26 Aircraft Acoustical Enclosure Restraint System
Proof Load/Pull Angles...................................... 126

BIBLIOGRAPHY.. 127

REFERENCES.. 128

Section 1: INTRODUCTION

1.1 Scope. This handbook includes information and criteria for designing rigid airfield pavements for all Navy and Marine Corps airfield facilities. These criteria encompass subgrade soils, base course materials, stabilized layers, concrete materials, aircraft loadings, and rigid pavement thickness design. The rigid pavement thickness design is based on the Westergaard theory of a slab loaded at the interior resting on a dense liquid foundation. Criteria are also presented for strengthening existing rigid pavements.

1.2 Cancellation. This military handbook, MIL-HDBK-1021/4, Rigid Pavement Design For Airfields, cancels and supersedes Chapter 4 (Section 5) and Chapter 5 (Section 3) of NAVFAC DM-21, Airfield Pavements, June 1973.

1.3 Related Criteria. Additional criteria related to the design of rigid airfield pavements may be found in the following applicable publications:

Subject	Source
Architecture	
Noise criteria	DM-1.03
Civil Engineering	
Pavements	DM-5.04
Soils and Foundations	
Soil mechanics	DM-7.01
Foundations and earth structures	DM-7.02
Airfield Pavement Design	
Airfield pavement design, evaluation, and marking; soil stabilization for pavement; design for frost conditions and subsurface drainage; skid-resistant runway surfaces	MIL-HDBK-1021/1 MIL-HDBK-1021/2 DM-21.03 DM-21.06 DM-21.09 Army TM 5-822-4
Petroleum Fuel Facilities	
Direct fueling stations for fixed-wing aircraft and helicopters; fuel distribution; dispensing fuel to aircraft and surface vehicles; operating fuel storage	DM-22
Airfield Lighting	MIL-HDBK-1023/1

Navigational and Traffic Aids	NAVAIR 51 50-AAA-2
Maintenance Facilities	
Hangars; power check pad; boresight range; weapons alignment facility	MIL-HDBK-1028/1 MIL-HDBK-1028/6

Section 2: DESIGN INPUT DATA

2.1 General. This section briefly identifies the input data required for rigid airfield pavement design. Later sections will discuss the design requirements and data collection.

2.2 Subgrade Soils. Subgrade soil properties are required to determine the strength of the subgrade and its load-bearing capacity. Perform a soil investigation to obtain the physical and engineering properties of the subgrade soil at the site. Conduct soil tests to determine the soil classification, strength, and the modulus of subgrade reaction (k value). Subgrade evaluation and determination of the subgrade k value are discussed in Section 3.

2.3 Base Course Materials. The gradation and durability of granular base materials is needed to evaluate the drainability and soundness of the aggregate. The k value on top of the base material is required for rigid airfield pavement thickness design. A graph is provided in Section 5 to determine the effective k value on top of a granular base.

2.4 Concrete Materials. Base the concrete mix design on the strength and durability of concrete paving materials. Section 6 discusses concrete materials including aggregates, admixtures, hot and cold weather considerations, and concrete mix design.

2.5 Climate and Drainage Considerations. Evaluate the drainability and frost susceptibility of the subgrade and base materials. Sections 3, 4, and 5 discuss drainage and frost considerations.

2.6 Traffic Data. Traffic information is required to design the concrete slab thickness to serve the projected traffic volumes and loads over the design life of the pavement. To determine the required pavement thickness, determine the type of aircraft and design gear loads, and forecast the total number of passes of each aircraft that is expected to use the pavement feature over its design life. Sections 7 and 8 discuss traffic and pavement thickness design.

2.7 Existing Pavement Condition. When designing overlays of rigid airfield pavements, the present stage of deterioration must be known in order to evaluate and apply cost-effective rehabilitation. Perform a condition survey to determine the types and severities of distress present in the pavement. Concrete thickness and strength and effective k value data must also be obtained. Nondestructive deflection testing can be used to determine the existing properties of the pavement. Input data for overlay design are discussed in Section 11.

PAGE 4 INTENTIONALLY BLANK

Section 3: SUBGRADE EVALUATION AND PREPARATION

3.1 General. Perform a subgrade evaluation to determine the soil classification and strength of the subgrade. The subgrade evaluation furnishes information about the soil type, its load-bearing capacity, and its expected drainability. This data will aid in evaluating the extent to which the subgrade soil can provide a sufficiently strong and stable foundation for the pavement being designed.

3.2 Soil Investigation. Perform a soil investigation to obtain the physical and engineering properties of the in situ soil. A soil investigation should consist of a soil survey, soil sampling, testing of soil samples, and a survey of available construction materials. Obtain additional information such as climatic data, site topography, and soil maps to aid in the evaluation of soil conditions at the site.

3.2.1 Soil Survey and Sampling. Sampling and surveying procedures are outlined in ASTM D420. Prior to the actual soil survey, obtain previous soil investigations, evaluation reports, construction records, soil maps and reports, aerial photographs, and topographic maps to provide background information on the soil conditions. The field survey consisting of sampling and testing can then be conducted to aid in characterizing the physical and engineering properties of the soil. Borings are usually obtained for this purpose. Requirements for the spacing and depth of exploratory borings are given in Table 1. These are general guidelines and more tests may be required based on local experience or to account for any unusual features. An adequate number of borings must be taken to locate all important soil variations. Areas of uniform soil conditions may require fewer borings than areas with variable soil conditions or special problems such as swelling soils. Subgrade sampling and testing standards are given in Table 2.

3.2.2 Soil Tests. Soil tests are necessary to determine the physical and engineering properties. Standardized field and laboratory tests are summarized in Table 3.

3.3 Soil Classification System. The Unified Soil Classification System, ASTM D2487, is used to classify soils for engineering purposes. The Unified Soil Classification System is utilized because it permits reliable classification on the basis of a few inexpensive tests, and is the most common system in use. Soil characteristics for the classification groups are presented in Table 4. A rapid method for visual field classification for the Unified System is outlined in ASTM D2488.

3.4 Soil Strength Tests. Use subgrade strength tests in conjunction with soil classification to provide a comprehensive indication of the subgrade behavior and performance. Subgrade soil strength for rigid pavement design is based on the plate load-bearing test.

Table 1
Minimum Requirements for Spacing
and Depth of Exploratory Borings

Item	Spacing Requirements
Runways and Taxiways < / = 200 feet (61 m) wide	200 feet (61 m) on center longitudinally, on alternating sides of the centerline
Runways > 200 feet (61 m) wide	three borings every 200 feet (61 m) longitudinally (one boring on the centerline and one boring on each side of the centerline near the edges)
Parking aprons and pads	one boring per 10,000 square foot (929 square meter) area

Item	Depth Requirements
Cut areas	to a minimum of 10 feet (3 m) below finished grade
Shallow fill [areas where not more than 6 feet (2 m) of fill will be placed]	to a minimum of 10 feet (3 m) below existing ground surface
High fill areas	to 50 feet (15 m) below existing ground surface or to rock

Table 2
Subgrade Sampling and Testing Standards

DESCRIPTION	ASTM STANDARD
Borings	
Auger Samples	D1452
Split-Barrel Sampling	D1586
Thin-Walled Sampling	D1587
Density	
Sand-Cone Method	D1556
Nuclear Method (as a field check only)	D2922
Drive-Cylinder Method	D2937
Moisture	
Laboratory	D2216
Nuclear Method (in place-as a field check only)	D3017

Table 3
Subgrade Testing Standards

DESCRIPTION	ASTM STANDARD
Preparation of Soil Samples for Size Analysis	D421
Grain Size Analysis (Dry Sieving)	C136
Grain Size Analysis (Wet Sieving)	D422
Plastic Limit and Plasticity Index	D4318
Liquid Limit of Soils	D4318
Moisture Density Relation of Soils	D1557
Reducing Field Samples of Aggregates to Test Size	C702
Laboratory Determination of Moisture Content	D2216
Shrinkage Factors of Soils	D427
Permeability of Granular Soils	D2434

3.4.1 Determination of Modulus of Subgrade Reaction (k Value). Use ASTM D1196 method of testing to determine the modulus of subgrade reaction for the design of rigid pavements. The modulus of subgrade reaction (k value) is the ratio of unit pressure to deflection of a 30-inch (762 mm) diameter rigid steel plate. The modulus of subgrade reaction is calculated using the pressure (pounds per square inch) corresponding to a 0.05-inch (1.27 mm) deflection.

Table 4
Soil Characteristics Pertinent to Roads and Airfields

Major Divisions (1)	(2)	Symbol: Letter (3)	Symbol: Hatching (4)	Symbol: Color (5)	Soil Type (6)	Performance Value as Subgrade When Not Subject to Frost Action (7)	Performance Value as Subbase When Not Subject to Frost Action (8)	Performance Value as Base When Not Subject to Frost Action (9)
COARSE-GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW		Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	Excellent	Good
		GP		Red	Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	Good	Fair to good
		GM d		Yellow	Silty gravels, gravel-sand-silt mixtures	Good to excellent	Good	Fair to good
		GM u		Yellow		Good	Fair	Poor to not suitable
		GC		Yellow	Clayey gravels, gravel-sand-clay mixtures	Good	Fair	Poor to not suitable
	SAND AND SANDY SOILS	SW		Red	Well-graded sands or gravelly sands, little or no fines	Good	Fair to good	Poor
		SP		Red	Poorly graded sands or gravelly sands, little or no fines	Fair to good	Fair	Poor to not suitable
		SM d		Yellow	Silty sands, sand-silt mixtures	Fair to good	Fair to good	Poor
		SM u		Yellow		Fair	Poor to fair	Not suitable
		SC		Yellow	Clayey sands, sand-clay mixtures	Poor to fair	Poor	Not suitable
FINE-GRAINED SOILS	SILTS AND CLAYS LL IS LESS THAN 50	ML		Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor to fair	Not suitable	Not suitable
		CL		Green	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Poor to fair	Not suitable	Not suitable
		OL		Green	Organic silts and organic silt-clays of low plasticity	Poor	Not suitable	Not suitable
	SILTS AND CLAYS LL IS GREATER THAN 50	MH		Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor	Not suitable	Not suitable
		CH		Blue	Inorganic clays of high plasticity, fat clays	Poor to fair	Not suitable	Not suitable
		OH		Blue	Organic clays of medium to high plasticity, organic silts	Poor to very poor	Not suitable	Not suitable
HIGHLY ORGANIC SOILS		Pt		Orange	Peat and other highly organic soils	Not suitable	Not suitable	Not suitable

Note:

1. Column 3, division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is on basis of Atterberg limits; suffix d (e.g., GMd) will be used when the liquid limit is 25 or less and the plasticity index is 5 or less; the suffix u will be used otherwise.
2. In column 13, the equipment listed will usually produce the required densities with a reasonable number of passes when moisture conditions and thickness of lift are properly controlled. In some instances, several types of equipment are listed because variable soil characteristics within a given soil group may require different equipment. In some instances, a combination of two types may be necessary.
   a. Processed base materials and other angular materials. Steel-wheeled and rubber-tired rollers are recommended for hard, angular materials with limited fines or screenings. Rubber-tired equipment is recommended for softer materials subject to degradation.
   b. Finishing. Rubber-tired equipment is recommended for rolling during final shaping operations for most soils and processed materials.
   c. Equipment size. The following sizes of equipment are necessary to assure the high densities required for airfield construction:
      Crawler-type tractor — total weight in excess of 30,000 lb.
      Rubber-tired equipment — wheel load in excess of 15,000 lb. wheel loads as high as 40,000 lb may be necessary to obtain the required densities for some materials (based on contact pressure of approximately 65 to 150 psi).
      Sheepsfoot roller — unit pressure (on 6- to 12-sq-in. foot) to be in excess of 250 psi and unit pressures as high as 650 psi may be necessary to obtain the required densities for some materials. The area of the feet should be at least 5 percent of the total peripheral area of the drum, using the diameter measured to the faces of the feet.
3. Column 14, unit dry weights are for compacted soil at optimum moisture content for MIL-STD-621, method D 100, CE 55 compaction effort.
4. In column 15, the maximum value that can be used in design of airfields is, in some cases, limited by gradation and plasticity requirements.
   (Table V, MIL-STD-619B of 12 June 1968)

Table 4 (Continued)
Soil Characteristics Pertinent to Roads and Airfields

Potential Frost Action (10)	Compressibility and Expansion (11)	Drainage Characteristics (12)	Compaction Equipment (13)	Unit Dry Weight lb per cu ft (14)	Typical Design Values: CBR (15)	Typical Design Values: Subgrade Modulus k lb per cu in. (16)
None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	125-140	40-80	300-500
None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	110-140	30-60	300-500
Slight to medium	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	125-145	40-60	300-500
Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	115-135	20-30	200-500
Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	130-145	20-40	200-500
None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller	110-130	20-40	200-400
None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller	105-135	10-40	150-400
Slight to high	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	120-135	15-40	150-400
Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100-130	10-20	100-300
Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100-135	5-20	100-300
Medium to very high	Slight to medium	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	90-130	15 or less	100-200
Medium to high	Medium	Practically impervious	Rubber-tired roller, sheepsfoot roller	90-130	15 or less	50-150
Medium to high	Medium to high	Poor	Rubber-tired roller, sheepsfoot roller	90-105	5 or less	50-100
Medium to very high	High	Fair to poor	Sheepsfoot roller, rubber-tired roller	80-105	10 or less	50-100
Medium	High	Practically impervious	Sheepsfoot roller, rubber-tired roller	90-115	15 or less	50-150
Medium	High	Practically impervious	Sheepsfoot roller, rubber-tired roller	80-110	5 or less	25-100
Slight	Very high	Fair to poor	Compaction not practical	-	-	-

3.4.2 Number of Tests. Conduct a sufficient number of plate-bearing tests to provide a valid estimate of the subgrade support. A minimum of two or three tests are usually required for each pavement feature under design. Conduct tests in the most representative areas. Consider changes in soil classification, cut and fill areas, and drainage conditions that may affect the k value in selecting the location of tests.

3.4.3 Correlations With Other Tests. Correlations have been developed for estimating subgrade support properties based on other tests and soil classification data. It is emphasized that correlations are not precise and should only be used for preliminary design. Figure 1 provides typical k values for soils based on the Unified Soil Classification System and other tests.

3.5 Drainage. Subsurface drainage must be considered in design. See NAVFAC DM-21.06, Airfield-Pavement Design for Frost Conditions and Subsurface Drainage, for subsurface drainage criteria.

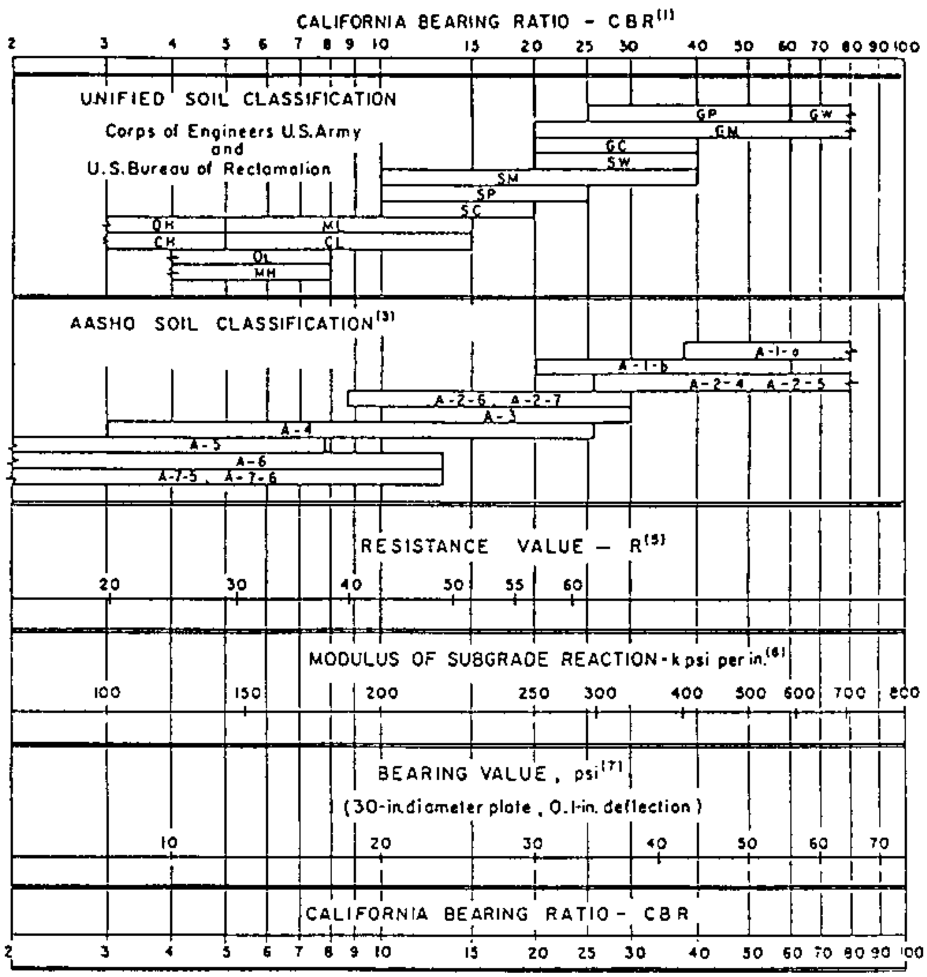
3.6 Special Problems.

3.6.1 Frost Heave. Fine-grained soil exposed to low temperature undergoes a process that involves the formation of ice lenses oriented roughly parallel to the surface. The individual ice lenses increase in thickness to several inches, and the frozen soil assumes the character of a stratified material consisting of alternate lenses of soil and clear ice. This process results in raising of the ground surface which is called frost heave. Significant frost heaving occurs primarily in fine-grained soils which allow excessive water movement.

In perfectly uniform soils, ice lenses do not develop unless the grains are smaller than 0.01 mm. In fairly uniform soils, ice lenses develop if at least 10 percent of the grains are smaller than 0.02 mm. In mixed-grained soils, ice lenses develop if at least 3 percent of the grains are smaller than 0.02 mm. In soil in which fewer than 1 percent of the grains are smaller than 0.02 mm, ice lenses do not develop.

Frost action problems can be minimized by eliminating excess water in the pavement structural section or by providing sufficient thickness of non-frost susceptible material. See NAVFAC DM-21.06 for procedures for design for frost conditions.

3.6.2 Swelling Soils. The swelling potential of a soil is directly related to its plasticity index. Typically, a soil possessing a plasticity index greater than 20 has a high potential for swelling. Consider the following factors when working with soils with high swelling potential.



(1) For the basic idea, see O.J. Porter, "Foundations for Flexible Pavements," Highway Research Board Proceedings of the Twenty-second Annual Meeting, 1942, Vol. 22, pages 100-136.

(2) "Characteristics of Soil Groups Pertaining to Roads and Airfields," Appendix B, The Unified Soil Classification System, U.S. Army Corps of Engineers, Technical Memorandum 3-357, 1953.

(3) "Classification of Highway Subgrade Materials," Highway Research Board Proceedings of the Twenty-fifth Annual Meeting, 1945, Vol. 25, pages 376-392.

(5) F.N. Hueem, "A New Approach for Pavement Design," Engineering News-Record, Vol. 141, No. 2, July 8, 1948, pages 134-139. R is factor used in California Stabilometer Method of Design.

(6) See T.A. Middlebrooks and G.E. Bertram, "Soil Tests for Design of Runway Pavements," Highway Research Board Proceedings of the Twenty-second Annual Meeting, 1942, Vol. 22, page 152. k is factor used in Westergaard's analysis for design of concrete pavement.

(7) See items (6), page 184.

Figure 1
Relationship of Soil Classifications and Bearing Values to k Values

3.6.2.1 Moisture Content. The most important factor governing soil swelling characteristics is the difference between the initial field moisture content at the time of construction and the moisture content that will develop after the pavement is in place. Significant change in soil moisture content results in a soil volume change. In addition, proper drainage is very important to remove excess water.

3.6.2.2 Degree of Compaction. Consider the degree of compaction of the fill or the degree of overconsolidation of natural soil. Typically, higher compaction levels in fill areas or previously high overburden pressures favor swelling as the soil increases in moisture.

3.6.2.3 Plasticity. Swell potential is also related to the percentage of material in the clay fraction, the fineness of the clay fraction, the clay structure, and the type of clay mineral. Consider a soil with an in situ moisture content approaching the plastic limit as having a high swell potential.

3.6.2.4 Construction Considerations. If there is only a thin layer of swelling soil, it may be economically feasible to remove it. Otherwise, consider lime stabilization to control swell of the soil. Lime stabilization is discussed in Section 4 and Department of the Army TM 5-822-4, Soil Stabilization For Pavements.

3.6.3 Variability Through Project Area. Over the length and width of a given project, the soil type, moisture content, strength, etc., are usually not constant. Variability is a natural phenomenon.

There are several ways to determine design values for a project site that possesses substantial soil variability. These include the average value of tests plus or minus (depending on the test) one to two standard deviations or using the most conservative values. The procedure to be used is a matter of engineering judgment and will depend on the test values.

3.7 Stabilization. Stabilization is a process of improving the properties of the existing material through the addition of a chemical or physical binder. Stabilization mechanisms, suitable material for each stabilizer, and construction techniques are summarized in Section 4. Detailed design criteria are given in TM 5-822-4.

3.8 Imported Fill. An alternative to soil stabilization is to excavate the unacceptable material and replace it with higher-quality material. If adequate borrow material is located close enough to the project site, it may be the most cost-effective alternative. Conduct exploration and testing of borrow material as described in NAVFAC DM-7.01, Soil Mechanics.

3.9. Density and Compaction. Compacting the soil to the maximum density at optimum moisture content is beneficial for minimizing future settlement, maximizing shear strength, and minimizing permeability. The exception to this is when swell potential exists in the subgrade soil and a moisture content other than optimum may be chosen during compaction.

The moisture-density laboratory testing procedure commonly used in pavement design and construction is the modified Proctor test. The modified Proctor test is described in ASTM D1557, Method D. Minimum subgrade compaction for the top 12 inches (305 mm) should be 100 percent of the modified Proctor maximum density at optimum moisture for cohesionless sand and gravel and 95 percent for all other soils. Minimum compaction for the subgrade below 12 inches (305 mm) should be 95 percent of the modified Proctor maximum density at optimum moisture for cohesionless sand and gravel and 90 percent for all other soils.

Field tests for density are given in Table 2 and include the sand cone method (ASTM D1556), nuclear density gauge method (ASTM D2922) and the drive cylinder method (ASTM D2937).

PAGE 14 INTENTIONALLY BLANK

Section 4. SOIL STABILIZATION

4.1 Scope. Stabilization is the process of improving the properties of the soil through the addition of a chemical or physical binder. This section provides guidelines for improving subgrade soil properties to expedite construction and improve subgrade strength and durability through the use of stabilizing admixtures. See TM 5-822-4 for a detailed discussion on soil stabilization.

4.2 Objectives. Consider soil stabilization during the design stage for the following potential benefits.

4.2.1 Construction Expediency. Stabilization techniques can be used to expedite construction in unsuitable conditions by improving resistance to deformation, load-carrying capacity, and shear strength. These improvements provide an efficient, effective, and economical working platform for construction operations.

4.2.2 Subgrade Modification. Stabilization can be used to improve workability, decrease plasticity, reduce swell, increase support, and increase resistance to the detrimental effects of moisture and freeze-thaw cycles.

4.2.3 Strength and Durability Improvements. Stabilization techniques can be used to significantly increase the strength and durability of soils to provide an economical paving material.

4.3 Types of Stabilizers. Stabilizers can be classified as active or inert. Active stabilizers react chemically with the soil or aggregate to produce the desired changes in engineering characteristics. Soil chemistry and mineralogy are important to the reaction between the admixture and the soil. Lime, Portland cement, and lime-fly ash are all active stabilizers.

Inert stabilizers do not react chemically with the soil or aggregate. Stabilization benefits of inert stabilizers are derived from the presence of the admixture and the physical binding of the admixture and the soil particles. Asphalt materials are inert stabilizers. The mechanical blending of in-place soils without an admixture can be an effective means of improving the subgrade for construction expediency and increased resistance to freeze-thaw effects. However, due to the highly variable nature of subgrade soils, the benefits of blending are likely to be highly variable throughout a given project area.

4.4 Criteria for Requiring Soil Stabilization. Base the decision to specify soil stabilization on a review of soil conditions necessary for construction or pavement design and performance. Consider stabilization if there is poor drainage, frost problems, the need for added subgrade strength, or the need for an improved working platform. Compare the cost of alternative solutions such as importing suitable granular materials to the cost of stabilization. Criteria for selecting a stabilizing agent are outlined in TM 5-822-4.

4.5 Lime Stabilization. Lime stabilization is used primarily for fine-grained soils such as clays.

4.5.1 Stabilization Mechanisms. The following reactions take place when lime is added to the soil:

4.5.1.1 Cation Exchange and Flocculation and Agglomeration. Cation exchange and flocculation and agglomeration reactions occur immediately and produce changes in soil plasticity, workability, and uncured strength properties.

4.5.1.2 Soil-Lime Pozzolanic Reaction. A soil-lime pozzolanic reaction resulting in the formation of various cementing agents may occur depending on the soil characteristics. This reaction is time-dependent and temperature-dependent; therefore, the strength gain is gradual but continuous over long periods of time. The factors which influence the soil-lime pozzolanic reaction are: time, temperature, lime type, lime content, and soil properties, such as organic carbon content, clay content, clay mineralogy, soil pH, natural drainage, and carbonate content.

4.5.2 Suitable Soils for Lime Stabilization. Lime can be used with medium- and fine-grained soils. Clays and silts with a plasticity index of 12 or greater with at least 12 percent of the material passing the No. 200 sieve can be considered for lime stabilization. Laboratory tests are necessary to determine the effectiveness of lime stabilization and to develop a mixture design. See TM 5-822-4 for additional criteria on when to use lime stabilization and procedures to determine the initial design lime content.

4.5.3 Lime Stabilization Construction Sequence. To assure that the desired stabilization objectives are met, the following construction sequence should be followed:

a) shape, scarify, and pulverize,
b) add lime,
c) mix and pulverize,
d) compact,
e) water cure,
f) scarify and pulverize,
g) compact,
h) cure.

The lime is generally mixed with the pulverized soil and allowed to cure for a period of 3 to 14 days to assist in breaking down heavy clay soils.

4.5.4 Types of Lime Application. Consider the advantages and disadvantages of the available lime application procedures.

4.5.4.1 Dry Hydrated Lime. Dry hydrated lime is faster to apply than a slurry and is effective in drying out soils. The dry hydrated lime may produce a dusting problem and the fast drying action may require an excess amount of water during dry hot weather.

4.5.4.2 Quicklime. Quicklime may be more economical than dry hydrated lime because the quicklime contains approximately 25 percent more available lime. Quicklime also provides faster reaction and drying with soils. Quicklime may produce a coarse material, requires more water which may cause problems in dry areas and presents greater susceptibility to skin and eye burns. Quicklime is not recommended unless safety requirements are met for handling.

4.5.4.3 Slurry Lime. Slurry lime provides a dust-free application, better distribution and minimizes drying action. Slurry lime is not practical for use with very wet soils.

4.5.5 Quality Control and Laboratory Tests. See TM 5-822-4 for information on strength and durability requirements, mixture design, and the required tests and laboratory procedures for lime stabilization. The minimum acceptable 28-day unconfined compressive strength for lime treated soils is 200 pounds per square inch (1 379 000 Pa). In addition to the quality of the soil-lime mixture, consider the fineness and purity of the lime. The fineness will affect the reactivity and mixing and distribution of the lime while the purity will affect the amount of reactive material available in the lime. Suitable grades of lime will have 95 percent or more passing the No. 200 sieve. See ASTM C207 for specifications on acceptable lime grades.

4.6. Portland Cement Stabilization. Cement stabilization is used primarily with coarse-grained materials such as crushed stones, gravels, and sands. Cement stabilization can be used to modify the plasticity of fine-grained soils; however, this practice is usually uneconomical because of increased cement requirements. However, if lime is not locally available, modifying fine-grained soils with cement may be an acceptable alternative.

4.6.1 Stabilization Mechanisms. Increases in strength and durability of the soil-cement mixture result from the formation of cementing agents due to the hydration of the Portland cement. Additional benefits such as improved workability may also be derived from a pozzolanic reaction if fines are present, and from cation exchange, flocculation and agglomeration when free lime is present in the cement.

4.6.2 Suitable Soils for Portland Cement Stabilization. Portland cement stabilization is best suited for well-graded granular materials. Poorly graded sands and low-plasticity clays may also be stabilized with cement although a higher cement content will normally be required and pulverization and mixing may be difficult. In general, suitable soils for cement stabilization should have a plasticity index of less than 20 and a minimum of 45 percent passing the No. 4 sieve. See TM 5-822-4 for additional criteria on when to use Portland cement stabilization.

4.6.3 Portland Cement Stabilization Construction Sequence. The objective in soil cement stabilization is to thoroughly mix the soil and cement at the required moisture content to allow for compaction and curing. The following construction sequence for in-place mixing should be followed:

a) shape, scarify, and pulverize,
b) add cement,
c) add water,
d) mix,
e) compact,
f) cure.

4.6.4 Quality Control and Laboratory Tests. See TM 5-822-4 for information on gradation requirements, mixture design, and laboratory testing procedures.

4.7 Asphalt Stabilization. Asphalt stabilization is used primarily with well to poorly graded sands, gravels, or crushed stone.

4.7.1 Stabilization Mechanisms. Asphalt stabilization is a form of inert stabilization. Improvements in soil properties are due to the presence of the asphalt material and the physical bonding and waterproofing of particles. Emulsions can be used for mixed-in-place stabilization. Environmental restrictions on cutbacks may preclude their use for mixed-in-place stabilization.

4.7.2 Suitable Materials for Asphalt Stabilization. Asphalt stabilization is suitable for granular materials such as sands and gravels with low fines and plasticity. Generally, soils with less than 30 percent passing the No. 200 sieve and a plasticity index of less than 10 can be stabilized with asphalt materials. See TM 5-822-4 for additional criteria for asphalt materials stabilization.

4.7.3 Construction Sequence. To ensure that the stabilization objectives are met, the following construction sequence should be followed:

a) shape, scarify, and pulverize,
b) add stabilizer,
c) mix,
d) aerate,
e) compact.

Aeration is required to allow the water or cutback solvent to evaporate and reduce the amount of fluids. Excessive fluids reduce the stability of the mix after it is compacted.

4.7.4 Quality Control and Laboratory Tests. See TM 5-822-4 for guidelines on acceptable asphalt materials and design procedures. The minimum recommended Marshall stability for the stabilized mixture is 500 pounds (227 kg).

4.8 Lime-Fly Ash Stabilization. A lime-fly ash mixture can be used for stabilization of silts, sands, gravels, and crushed stone. Fly ash is a by-product of the combustion of powdered coal. Typical lime-fly ash ratios range from 1:3 for a highly reactive fly ash to 1:5 for a poorly reactive fly ash. Typical lime-fly ash contents range from 12 to 15 percent for well graded gravels to 20 to 30 percent for sands and poorly graded materials.

Fly ash is a highly variable material and its quality is difficult to check using mineralogy; therefore, performance tests should be used. See ASTM C593 for fly ash and lime-fly ash mixture specifications and guidance in selecting fly ash. Generally, the loss-on-ignition should be less than 10 percent, and greater than 85 percent of the lime-fly ash mixture should pass the No. 325 sieve.

Lime-fly ash stabilization is acceptable in Navy and Marine Corps construction only for limited test or experimental purposes. Approval must be obtained from the Naval Facilities Engineering Command Headquarters.

4.9 Thickness of Stabilized Subgrade Layers. No reduction in pavement thickness is normally made when a stabilized subgrade is used. However, if the stabilized subgrade meets the requirements for a base course, and is at least 6 inches (150 mm) in thickness, then a 1:1 thickness replacement ratio for base course can be made. See Section 5.2 for base course material requirements and Section 5.3 for minimum base course thickness requirements.

PAGE 20 INTENTIONALLY BLANK

Section 5: BASE COURSE MATERIALS

5.1 Introduction. In rigid pavement design, the base course refers to the layer of material that lies directly beneath the concrete slab. The base may consist of more than one layer. The primary functions of the base may consist of the following:

a) provide uniform support to the pavement throughout the design life,
b) improve load transfer across joints,
c) prevent pumping by providing drainage through the use of a material with little or no fines,
e) protect against frost action by reducing frost penetration into the subgrade,
f) provide a construction platform to expedite construction,
g) protect against volume change in subgrade soils by providing an overburden on expansive soils to minimize swelling.

The main structural support element in a rigid pavement is the Portland cement concrete slab. The most important function of the base course material in a rigid pavement is to provide uniform long-term support to the slab with adequate drainage to prevent pumping and loss of support.

The base course must be constructed of quality material and properly designed to ensure a good foundation. If pumping and loss of support occur, the performance of the concrete slab will be reduced.

5.2 Materials. Suitable materials for base courses include natural, processed, manufactured, and stabilized materials which meet ASTM D2940. These are the most common types of base course materials. Select local materials if possible, and consider local experience and practices when selecting a base material.

5.2.1 Requirements for Granular Materials. The satisfactory performance of the base course material depends on several factors discussed below.

5.2.1.1 Gradation. To provide adequate drainage, the base course must contain little or no fines (material that passes the No. 200 sieve). Gradation requirements assure that the base course provides adequate stability and drainage under repeated loads. Crushed aggregates have greater stability than round-grained materials.

5.2.1.2 Wear Resistance. Aggregates suitable for base course material must have the ability to withstand abrasion and/or crushing. Do not use soft aggregates for base course material because they may break down into fines which will inhibit drainage. Use the Los Angeles abrasion test (ASTM C131) for determining aggregate abrasion resistance. Aggregates suitable for base course shall have a percentage loss in the Los Angeles abrasion test less than or equal to 40 percent.

5.2.1.3 Freeze-Thaw Durability. Do not use aggregates that are easily broken down under freeze-thaw cycles in frost areas. This property can be determined in the laboratory by alternately freezing and thawing saturated samples of coarse aggregate. Place full reliance on field performance records.

5.2.1.4 CBR Requirements. Base course material shall have a minimum bearing ratio of 30 at 95 percent of the maximum laboratory density as determined by ASTM D1883. See NFGS-02232, Select Material (Base Course for Rigid) (and) (Subbase Course for Flexible) Pavement, for specifications on base course material for rigid pavement.

5.2.2 Stabilized Bases. Stabilized bases (e.g., cement-aggregate mixtures) can be used to improve load transfer at joints in the slab and provide increased support to the pavement. Design stabilized bases in accordance with criteria in TM 5-822-4.

5.2.3 Lean Concrete Bases. Lean concrete mixtures may be used as base material to provide increased support and reduce pumping. They may also be more economical than stabilized bases. Lean concrete refers to a mixture composed of low-cost, locally available aggregates that may not meet specifications for normal concrete mixtures and an amount of Portland cement that is usually less than for normal concrete mixtures. Local aggregates, substandard aggregates, and recycled materials may all be used in lean concrete mixtures for base materials. When properly designed, these materials can provide a strong and erosion-resistant base.

Material specifications and gradation requirements for aggregates used in lean concrete mixtures are not as restrictive as those for aggregates used in normal concrete. Aggregate gradations should conform to one of the gradations given in Table 5. The aggregate materials should be free from any elongated or soft pieces, and dirt. Mix design for lean concrete bases is discussed in Section 6.

Any bond between the lean concrete base and the concrete slab to be placed on top must be prevented to retard reflective cracking. A bond breaking material such as a wax-based curing compound should be placed on top of all lean concrete base courses.

5.2.4 Recycled Concrete Bases. Recycled Portland cement concrete can serve as an aggregate for use in a granular base course or in recycled concrete base. The concrete must be properly crushed and sized to meet gradation requirements given in NFGS-02232.

5.2.5 Geotextile Fabrics. Geotextile fabrics may be considered for reinforcement of the subgrade to provide a working platform for base course construction and to separate the subgrade and base course to maintain the original base course gradation. See NAVFAC DM-7.01 and NAVFAC DM-5.04, Pavements, for design criteria on geotextile fabrics.

Table 5
Gradations for Lean Concrete Base Materials

Sieve Size (square opening)	Percentage by Weight Passing Sieve A	B	C
2 inch (50 mm)	100	-	-
1.5 inch (37.5 mm)	-	100	-
1.0 inch (25.0 mm)	55-85	70-95	100
0.75 inch (19.0 mm)	50-80	55-85	70-100
No. 4 (4.75 mm)	30-60	30-60	35-65
No. 40 (425 mm)	10-30	10-30	15-30
No.200 (75 mm)	0-15	0-15	0-15

5.3 Design Considerations.

5.3.1 Minimum Thickness Requirements. The minimum thickness requirements for base courses are listed in Table 6. The minimum thickness for granular materials is set for construction purposes. The additional base thickness required over clays and silts is to aid in preventing pumping. Consider experience with local aggregates and materials when selecting the base course thickness.

Table 6
Base Course Minimum Thickness Requirements

Base Material	Minimum Thickness
Granular Material	6 inches (152 mm)
Cement Stabilized	6 inches (152 mm)
Asphalt Stabilized	6 inches (152 mm)
Asphalt Concrete	4 inches (102 mm)
Lean Concrete Mixture	4 inches (102 mm)

Note: For subgrades classified as CH, CL, MH, ML, or OL, the minimum granular base course thickness shall be 8 inches (203 mm).

5.3.2 Effective k Value. The k value is a measure of the stiffness of the supporting material. The addition of base material will provide a k value on top of the base material that is higher than the k value directly on the subgrade. This effective k value on top of the base is used in the pavement thickness design procedures in Section 8.

The effective k value on top of the base is a function of the subgrade k value, base type, and the base thickness. The k value on top of the subgrade (before any subgrade stabilization) is determined as outlined in Section 3. It is not practical during the design phase to require a plate-loading test to determine the k value on top of the base because construction of test sections is costly; therefore, the effective k value on top of the base is estimated from Figure 2.

Figure 2 was developed for pit run gravel or stone meeting NFGS-02232. The minimum acceptable effective k value for use in design is 200 pounds per cubic inch (5 536 000 kilograms per cubic meter). The maximum allowable effective k value for use in design is 500 pounds per cubic inch (13 840 000 kg/mÀ3Ù).

5.3.3 Base Type. A stabilized base or lean concrete base may be used as a substitute at a 1:1.5 thickness replacement ratio for all or part of the granular base course thickness determined from Figure 2, within the limits of the minimum thickness requirements listed in Section 5.3. However, the k value used for design remains the same as that determined at the top of the granular base. The design k value is not increased due to the use of a stabilized base or lean concrete base. An unbonded stabilized or lean concrete base does not increase the effective k value greatly due to slippage between the slab and base.

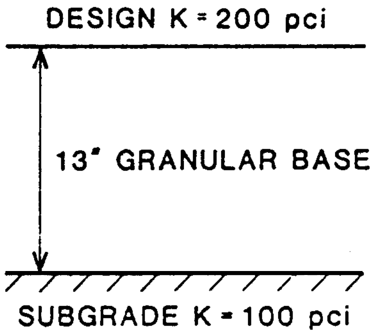
If the concrete design thickness is less than 9 inches (229 mm), a stabilized base course, of at least the minimum thickness given in Section 5.3, is required directly below the concrete pavement. If the concrete thickness is greater than or equal to 9 inches (229 mm), the base course may be granular material, stabilized material, or a combination of the two.

Stabilized subgrade may be used as a substitute for part of the aggregate base course at a 1:1 thickness replacement ratio if the stabilized subgrade meets the requirements for base course and is at least 6 inches (150 mm) in thickness. However, all concrete pavement designs shall include either a granular or stabilized base course.

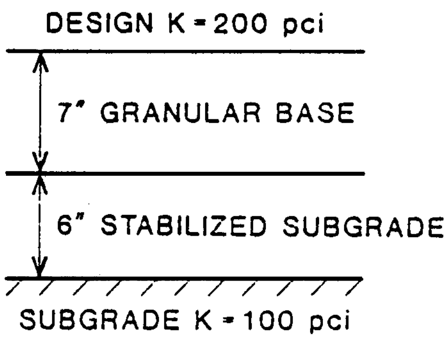
Example calculations: A concrete pavement is to be constructed on a clay (CH) subgrade. The k value for the existing subgrade as determined from plate load tests is approximately 100 pounds per cubic inch (2 768 000 kg/mÀ3Ù). The minimum design k value of 200 pounds per cubic inch (5 536 000 kg/mÀ3Ù) has been selected as the design k value. The granular base course thickness required to provide an effective k value of 200 pounds per cubic inch (5 536 000 kg/mÀ3Ù) on top of the base is determined by entering Figure 2 on the vertical scale with a k value of 200 pounds per cubic inch (5 536 000 kg/mÀ3Ù). A horizontal line is projected to the curve for a subgrade k value of 100 pounds per cubic inch (2 768 000 kg/mÀ3Ù). A vertical line is then projected down and the thickness of base course is determined. For this example, a 13-inch (330 mm) granular base course is required to provide an effective k value of 200 pounds per cubic inch (5 536 000 kg/mÀ3Ù) on top of the base.

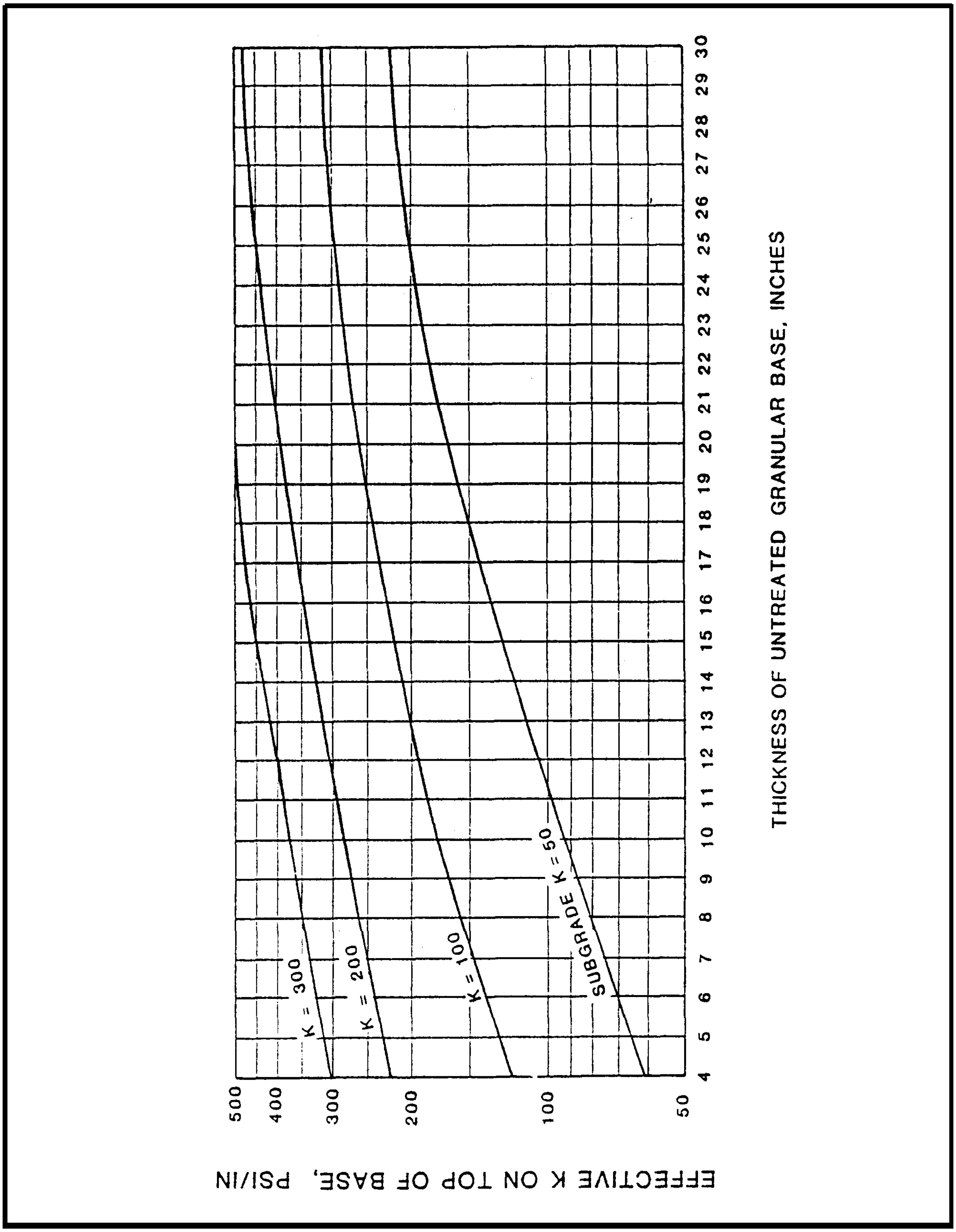
There are several acceptable base course designs for this example, depending on the required concrete thickness. If the concrete slab thickness is 9 inches (200 mm) or greater, the following alternatives are examples of acceptable designs:

a) Place a granular base course for the entire 13 inches (330 mm).

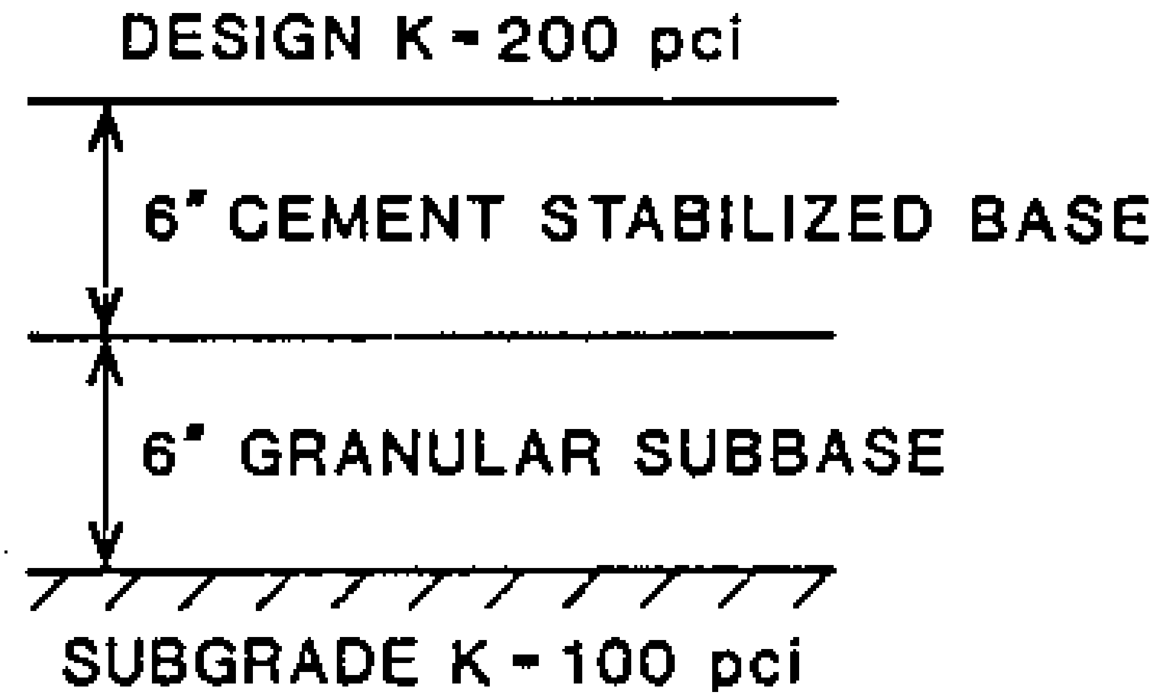


b) Substitute 6 inches (152 mm) of stabilized subgrade for 6 inches (152 mm) of the granular base course and construct a 7-inch (178 mm> granular base course.

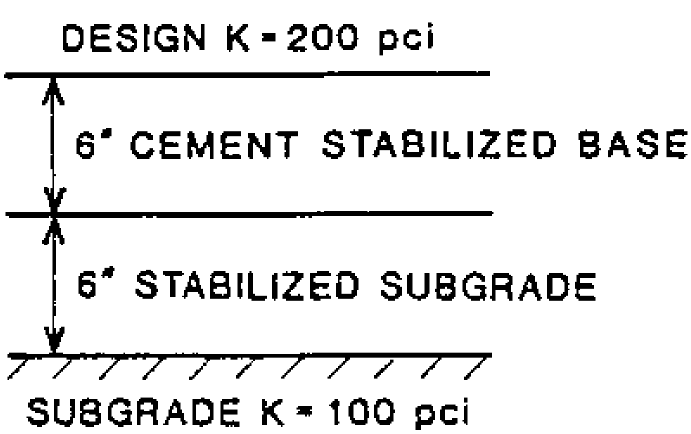



THICKNESS OF UNTREATED GRANULAR BASE, INCHES
4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30
EFFECTIVE K ON TOP OF BASE, PSI/IN
50 100 200 300 400 500
K = 300
K = 200
K = 100
SUBGRADE K = 50

c) Place a 6-inch (152 mm) granular subbase directly on top of the subgrade and substitute the remaining 7 inches (178 mm) of the granular material with a 6-inch (152 mm) cement stabilized base directly under the concrete pavement. A 6-inch (152 mm) granular subbase and 6-inch (152 mm) cement stabilized base are the minimum thicknesses.



d) Substitute 6 inches (152 mm) of the granular material with 6 inches (152 mm) of stabilized subgrade and substitute the remaining 7 inches (178 mm) of the granular material with a 6-inch (152 mm) stabilized base course.



5.3.4 Gradation. The gradation of the base course affects stability, drainage, and frost susceptibility. Base course gradations should meet NFGS-02232. In addition to the gradation requirements, the fine material passing the No. 40 sieve should have a plasticity index no greater than 5 and a liquid limit no greater than 25.

5.3.5 Drainage and Permeability.

5.3.5.1 Permeability. The permeability of granular materials is dependent upon grain-size distribution, grain shape, and relative density. To provide adequate drainage, the base course should be a well graded material, but it should contain little or no fines. For more detail on the design of drainage systems see NAVFAC DM-21.06. A filter layer should be used whenever a drainable base course is to be placed on a fine-grained subgrade and may be necessary on SM, SM-SC, and SC soil classifications.

5.3.5.2 Filter Layer. The filter layer protects a drainable granular base course from the intrusion of subgrade fines to maintain a drainable material and to minimize pumping. The gradation of a granular filter layer is controlled by the following criteria:

DÚ15¿FILTER/DÚ15¿SUBGRADE > / = 5

and

DÚ15¿FILTER/DÚ85¿SUBGRADE < / = 5

WHERE: DÚ15¿ = parcle size where 15 percent of the material is finer

DÚ85¿ = particle size where 85 percent of the material is finer

When protected subgrade soil is plastic and without sand or silt, the D15 of the filter need not be less than 0.039 inches (1 mm). The minimum filter thickness should be 4 inches (102 mm). Greater filter thicknesses may be required to provide a working platform during placement and compaction of the base course. Additionally, proper compaction of the subgrade will minimize the possibility of intrusion of fines through the filter and into the base. Geotextiles which meet the filter criteria may also be used.

5.3.6 Drainage Layer Beneath Stabilized Base. When a stabilized base is used and there is an excessive amount of free moisture available along with high volumes of heavy traffic, consider a drainage layer beneath the stabilized layer or otherwise provide subsurface drainage to minimize the potential for pumping.

5.4 Quality Control. Compact base materials to a high density to ensure adequate stability. To achieve high densities, control the moisture content and maintain it close to or slightly above the optimum moisture content. Clean granular materials may be compacted at higher moisture contents. The lift thickness of base course materials must also be controlled since high densities are difficult to achieve in thick lifts. Lift thickness for granular materials may range from 4 to 8 inches (102 to 203 mm) depending on the type of compaction equipment being used. Base course materials should be uniformly distributed over the subgrade to avoid segregation.

5.4.1 Compaction. Compact granular and cement-treated base courses to 100 percent of maximum density according to ASTM D1557 and D558, respectively. Compact asphaltic concrete base courses to 97 percent of the maximum density as determined from the Marshall mix design method.

5.4.2 Construction Procedure. The construction sequence for base courses includes the following general steps:

a) compact and grade subgrade,
b) spread base course material,
c) shape and compact base,
d) grade and compact to required elevation.

5.4.3 Testing Considerations. Graded aggregate base course materials must satisfy NFGS-02232. Test methods are summarized in Table 7.

TABLE 7
Test Methods for Base Course Materials

TEST	ASTM STANDARD
Sampling Materials	D75
Grain Size Analysis (Dry Sieving)	Cl36
Grain Size Analysis (Wet Sieving)	D422
Liquid Limit	D4318
Plastic Limit and Plasticity Index	D4318
Sand Equivalent Value	D2419
Percent of Wear	Cl31
Sand Cone Method	D1556
Nuclear Method (as field check)	D2922

5.4.1 Compaction. Compact granular and cement-treated base courses to 100 percent of maximum density according to ASTM D1557 and D558, respectively. Compact asphaltic concrete base courses to 97 percent of the maximum density as determined from the Marshall mix design method.

5.4.2 Construction Procedure. The construction sequence for base courses includes the following general steps:

a) compact and grade subgrade,
b) spread base course material,
c) shape and compact base,
d) grade and compact to required elevation.

5.4.3 Testing Considerations. Graded aggregate base course materials must satisfy NFGS-02232. Test methods are summarized in Table 7.

TABLE 7
Test Methods for Base Course Materials

TEST	ASTM STANDARD
Sampling Materials	D75
Grain Size Analysis (Dry Sieving)	C136
Grain Size Analysis (Wet Sieving)	D422
Liquid Limit	D4318
Plastic Limit and Plasticity Index	D4318
Sand Equivalent Value	D2419
Percent of Wear	C131
Sand Cone Method	D1556
Nuclear Method (as field check)	D2922

THIS PAGE IS INTENTIONALLY BLANK

Section 6: PORTLAND CEMENT CONCRETE MATERIALS

6.1 Introduction. This section provides guidelines for the selection of cement types, aggregates, and admixtures. Recommended aggregate and concrete testing procedures are also covered. See NFGS-02520, Portland Cement Concrete Pavement for Roads and Airfields, for additional information.

6.2 Heat Resistant Concrete for F/A-18 Aircraft. Pavement areas that are exposed to jet blast for an extended period of time may experience damage in the form of surface scaling. Surface scaling is a result of differential expansion and vaporization of moisture in the concrete. Aggregates having a low coefficient of thermal expansion and low porosity, such as basalt, will provide greater resistance to surface scaling than will carbonate aggregates. When designing parking apron pavements for the F/A-18 provide heat resistant concrete in each parking position to withstand the heat from the auxiliary power unit (APU). The aggregate shall be trap rock, fine grained, of the diabase or basalt variety, free of voids, quartz, and zeolites, all as described in ASTM C294. The aggregates shall meet the requirements of ASTM C33 and shall be tested for reactivity with alkalies in accordance with ASTM C289. Fine aggregate shall be produced from the same material as the coarse aggregate.

6.3 Cement Types. See ASTM C150 for Portland cement specifications. The five general ASTM designations for Portland cement are listed below.

6.3.1 Type I. Use in general construction where no special properties are needed and where concrete is not exposed to sulfate attack.

6.3.2 Type II. Use when concrete is exposed to moderate sulfate action. Type II generates less heat than Type I during hydration and may be used for mass projects or when placing concrete during hot weather.

6.3.3 Type III. Use when high early strength is required. It is used when forms must be removed quickly and when a reduced curing period is necessary before opening to traffic. Type III generates more heat than Type I during hydration and may be used when placing concrete in cold weather.

6.3.4 Type IV. Use when low heat of hydration is required. Strength development is slower than Type I.

6.3.5 Type V. Use when concrete is exposed to extreme sulfate action.

Types IA, IIA, and IIIA are Portland cements into which an air-entraining addition is blended during manufacture. Do not use Types IA, IIA, and IIIA because of the inability to control air content and lack of uniformity at the concrete plant.

6.3.6 Criteria for Selection of Cement Types. The primary factors in selecting a cement are resistance to detrimental chemical actions (e.g., sulfate attack), construction considerations (e.g., temperature at time of construction, rapid strength gain), and availability.

6.3.6.1 Chemical Actions. If the concrete will be exposed to sulfates, choose a sulfate-resistant cement. Table 8 describes sulfate concentrations and the recommended cement types.

6.3.6.2 Construction Considerations. The temperature at the time of construction may dictate the selection of the cement type. If construction is performed in cold weather, a Type III cement may be used to generate a greater amount of heat during hydration and accelerate hardening. If construction is performed in hot weather, a Type IV cement may be used to reduce the heat of hydration and slow hardening. If rapid strength gain is required in order to return a facility to service as soon as possible, a Type III cement may be used to provide rapid hardening and high early strength gain.

6.3.6.3 Availability. The availability of a cement type may dictate which cement type is chosen. Types I and III are generally readily available throughout the United States. Type II is common in many western states where the soils contain a greater sulfate concentration. Types IV and V may only be available in areas where a special market exists and they are routinely used.

Table 8
Types of Cement Required for Concrete
Exposed to Sulfate Attack

Relative Degree of Sulfate Attack	Percentage Water-Soluble Sulfate (as SOÚ4¿) in Soil Samples	Sulfate (as SOÚ4¿) in Water Samples (parts/million)	Cement Type
Negligible	0.00 to 0.10	0 to 150	I
Positive	0.10 to 0.20	150 to 1,500	II
Severe	0.20 to 2.00	1,500 to 10,000	V*
Very Severe	2.00 or more	10,000 or more	V+pozzolan**

Source: U.S. Bureau of Reclamation, Concrete Manual, 1979

* Or approved Portland-pozzolan cement providing comparable sulfate resistance when used in concrete.
** Should be approved pozzolan that has been determined by tests to improve sulfate resistance when used in concrete with Type V cement.

6.4 Concrete Aggregates. Aggregates have an important influence on the properties of Portland cement concrete. Aggregates are an economical filler and provide concrete with dimensional stability and wear resistance. The following sections provide information on aggregate types, recommended gradations, performance, and the use of recycled Portland cement concrete for use in a surface concrete mix.

6.4.1 Aggregate Types. Aggregates are classified as light weight, normal weight, or heavy weight. This discussion is limited to normal weight aggregates which are used in the majority of pavement construction.

Aggregates must be hard and strong, inert, and free from any impurities such as silt, clay, or organic matter. Soft aggregates can limit the strength and reduce durability of the concrete. Impurities may increase water requirements and interfere with the hydration reactions.

The aggregate shape and texture affect the workability of fresh concrete and the strength of the hardened concrete. Angular, elongated, or irregular shaped aggregates will increase paste requirements because of increased interparticle interaction. Elongated aggregates may also cause problems with segregation during handling. While rough, textured surfaces may increase mechanical bond, an elongated shape may also indicate an aggregate with weak fracture planes. This may have an adverse effect on the hardened concrete strength.

Consider the durability of the aggregate when designing a concrete mix. Both the chemical and physical durability of the aggregate must be considered. Physical durability concerns the soundness, wear resistance and freeze-thaw characteristics of the aggregate. See NFGS-02520 for specifications on soundness, wear resistance and freeze-thaw damage.

Chemical durability concerns the cement-aggregate reactivity. Reaction of alkali in cement with siliceous or carbonate aggregates causes expansion damage to the concrete, resulting in map cracking. Sedimentary rocks such as limestone, shale, and sandstone which contain amorphous silica such as opal and chert are susceptible to reactivity with alkalies in cement. The alkali-silica reaction can be controlled with a pozzolanic admixture. Some carbonate rocks such as dolomitic limestones can also react with alkali in cement and cause expansive damage to concrete. The alkali-carbonate reaction cannot be controlled with pozzolans. Specify cements low in alkali content for use with aggregates susceptible to alkali-carbonate reactivity. See ASTM C227 and ASTM C289 for specifications on alkali reactivity.

6.4.2 Gradation. Aggregate gradation influences the amount of cement that is required, the handling characteristics of the plastic concrete, and the hardened concrete properties. See ASTM C33 for gradation limits for coarse and fine aggregates.

6.4.3 Maximum Aggregate Size. For pavement slabs, the maximum aggregate size should not exceed 1.0 to 1.5 inches (25 to 38 mm). If reinforcement is used, the maximum size should not exceed three fourths of the minimum clearance between reinforcing bars or reinforcing bars and forms.

6.4.4 Performance of Local Aggregates. Use suitable local aggregates if available. This should result in lower costs for transporting materials. Thoroughly test aggregates for resistance to "D" cracking, reactivity, soundness, and abrasion resistance. Primary concerns with local aggregates center around "D" cracking and reactivity.

6.4.4.1 "D" Cracking. Durability or "D" cracking refers to deterioration along a joint or crack in a slab caused by the concrete's degradation due to freeze-thaw cycles. "D" cracking is related to aggregates that are expansive when saturated with water and then frozen. The pressure exerted by the repeated expansion is great enough to fracture the concrete and cause "D" cracking. "D" cracking usually begins at joints and cracks because free moisture is present and will appear as a pattern of cracks running parallel to the joint or linear crack. A dark coloring can usually be seen around fine durability cracks. Cracking may eventually lead to disintegration of the concrete within 1 to 2 feet (305 to 610 mm) of the joint or crack. Local aggregates must be tested for soundness and durability to ensure acceptable performance in areas where many cycles per year of freeze-thaw of the concrete slab occurs. Aggregates must satisfy ASTM C88 and ASTM C666. If aggregates are susceptible to "D" cracking the maximum aggregate size should be 1.0 inch (25 mm).

6.4.4.2 Reactive Aggregates. The most common chemical reactivity problem is the alkali-aggregate reaction. This reaction can take the form of an alkali-silica reaction or an alkali-carbonate reaction. The alkali-aggregate reaction results in the formation of a gel which is accompanied by a volume expansion. This volume expansion is sufficient to cause cracking throughout the concrete slab. The primary factors that control alkali-aggregate expansion are the presence of reactive silica, dolomite, or calcite; particle size of reactive material, amount of reactive alkali in cement, and amount of available moisture. Local aggregates must be evaluated for alkali-aggregate reactivity and satisfy ASTM C289.

6.4.5 Recycled Portland Cement Concrete. Recycled Portland cement concrete pavement can serve as a high-quality and low-cost aggregate in Portland cement concrete. The recycled concrete must be properly crushed and sized to meet aggregate gradation requirements.

Once the material has been properly sized, it must be tested for quality as outlined in this section. Recycled concrete aggregate meeting all quality requirements for new aggregate can be used in Portland cement concrete pavement. Lower-quality recycled concrete aggregate can be considered for use in base courses.

6.5 Admixtures. An admixture is a material other than water, aggregate, or cement that is added to the concrete batch immediately before or during mixing. Admixtures can be used to improve the handling and consolidation of plastic concrete or the performance and material characteristics of hardened concrete. Table 9 summarizes the principal advantages and disadvantages of the major types of admixtures. The major types of admixtures are described below. See NFGS-02520 for additional information and specifications.

Table 9
Admixture Effects on Concrete Properties

ADMIXTURE TYPE	FRESH CONCRETE	HARDENED CONCRETE
Air-Entraining	improved workability reduced bleeding reduced segregation	reduced strength improved freeze-thaw resistance improved sulfate resistance
Water-Reducing	improved workability may increase bleeding increase entrained air above level desired	increased strength increased impermeability improved durability
Set-Retarding	reduced workability increase entrained air above level desired	increased shrinkage reduced early strength
Set-Accelerating	-	increased early strength may decrease final strength increased shrinkage reduced sulfate resistance reduced alkali-aggregate resistance
Pozzolanic Material	improved workability	improved sulfate resistance increased impermeability improved durability increased strength

6.5.1 Air-Entraining Admixtures. Use air-entraining admixtures to improve workability and freeze-thaw durability and provide better overall resistance to sulfate actions. Air entrainment also reduces bleeding and segregation that may occur during transportation and handling. Entrained air is required in all Navy and Marine Corps concrete pavements. See ASTM C260 for specifications on air-entraining admixtures.

6.5.2 Water-Reducing Admixtures. Use water-reducing admixtures to lower the water required to attain a given slump. This generally leads to an increase in strength, impermeability, and durability. Possible disadvantages include an increase in bleeding and air entrainment. The active ingredient in water-reducing compounds is adsorbed at the solid-water interface. This neutralizes the surface charge on the particles so that all particle surfaces carry a like charge and repel each other. The particles are fully dispersed in the cement paste and free the water to reduce the viscosity of the cement paste. See NFGS-02520 and ASTM C494 for specifications on water-reducing admixtures.

6.5.3 Set-Retarding Admixtures. Use set-retarding admixtures to delay setting time when placing concrete in high temperatures, when delays may be unavoidable, or when placing large volumes of concrete. Set-retarding admixtures act to decrease the rate of early hydration. Possible detrimental effects on concrete properties include an increase in the rate of loss of workability even though the setting time is extended, a reduction in the early strength of the concrete, and an increase in shrinkage and creep. See NFGS-02520 and ASTM C494 for specifications on set-retarding admixtures.

6.5.4 Set-Accelerating Admixtures. Use set-accelerating admixtures to facilitate early strength gain and to overcome slow hydration rates due to low temperatures. The primary mechanism of accelerators is to increase the rate of the early stages of hydration. This results in increased 6-hour to 24-hour flexural strengths; however, strength gains diminish with time and the final flexural strength of the mix may be less than a concrete without the accelerating admixture. Accelerating admixtures may also increase shrinkage and creep. If calcium chloride is used, resistance to sulfate attack may be reduced and the alkali-aggregate reaction may be aggravated. See NFGS-02520 and ASTM D98 for specifications on set-accelerating admixtures.

6.5.5 Mineral Admixtures. Mineral admixtures can be subdivided into the following general categories.

6.5.5.1 Materials of Low Reactivity. These materials are used to increase the workability of plastic concrete which is deficient in fines. Common materials are ground limestone, dolomite, quartz and rock dust. Do not use these types of mineral admixtures because they seldom contribute to strength and durability of the hardened concrete.

6.5.5.2 Cementitious Materials. Use these materials to improve the strength of hardened concrete. Common materials are natural cements, hydrated lime, and blast furnace slag.

6.5.5.3 Pozzolanic Materials. Use these materials to improve the workability of plastic concrete, the impermeability and durability of hardened concrete. The addition of a pozzolan also improves the resistance to sulfate attack and alkali-silica reactivity. Common materials include natural pozzolans and synthetic pozzolans. Natural pozzolans such as chert and shale are seldom used because grinding to cement fineness is required. Synthetic pozzolans include fly ash and boiler slag. Fly ash is most commonly used because it is finely divided and can improve the workability of the plastic concrete without a large increase in water requirements.

The Navy and Marine Corps do not allow fly ash to be substituted for Portland cement; however, concrete designs using fly ash as a substitute for Portland cement will be evaluated on a project by project basis and must be approved by the Naval Facilities Engineering Command Field Division. Pozzolans should conform to ASTM C618. See NFGS-02520 for guide specifications on pozzolans.

6.5.6 Factors in Selecting Admixtures. The primary factor in deciding when to use an admixture is the effect the admixture will have on the plastic and hardened properties of the concrete. Primary concerns may include workability, placing characteristics, and early strength development. Incorporate admixtures during the mix design stage in order to evaluate the effect of admixtures.

6.6 Testing Procedures. See NFGS-02520 for guide specifications on field testing and sampling.

6.6.1 Aggregate Tests. The following sections provide brief information on the major specification tests.

6.6.1.1 Sieve Analysis and Gradation. Particle size distribution has a major effect on the water requirements and handling properties of fresh concrete. See ASTM C136 for specifications on sieve analysis and ASTM C33 for gradation limits on coarse and fine aggregates.

6.6.1.2 Specific Gravity and Absorption. Specific gravity and absorption are necessary for calculating batch quantities for mix design. Absorption is also used to evaluate freeze-thaw durability. See ASTM C127 and C128 for specific gravity and absorption of coarse aggregates and fine aggregates.

6.6.1.3 Unit Weight. The unit weight of aggregates is required for computing the volume of voids and approximating the amount of cement paste required in mix design. See ASTM C29 for specifications on unit weight.

6.6.1.4 Surface Moisture. The amount of surface moisture on the aggregate must be known to control mix design. ASTM C70 contains specifications for the measurement of surface moisture.

6.6.1.5 Soundness. Use soundness tests to evaluate the resistance to disintegration in aggregates due to cycles of wetting and drying or freezing and thawing. See ASTM C88 for specifications on the soundness tests. ASTM C666 provides specifications for freezing and thawing tests on aggregates.

6.6.1.6 Resistance to Abrasion. Abrasion tests are used to measure the deterioration or degradation of aggregates due to impact and surface abrasion. See ASTM C131 and C535 for specifications on abrasion tests.

6.6.1.7 Cleanliness and Deleterious Substances. Tests for deleterious substances and impurities are listed in Table 10.

Table 10
Tests for Deleterious Substances

TEST DESCRIPTION	ASTM DESIGNATION
Test for organic impurities in sands	C40
Effect of organic impurities on strength	C87
Test for lightweight pieces	C123
Test for clay lumps and friable materials	C142

6.6.2 Concrete Tests. Concrete tests and specifications are necessary to insure that hardened concrete with the required properties is attained. These tests can be broken down into tests for fresh concrete and tests for hardened concrete. The following sections provide some limited information on the major specification tests. See NFGS-02520 for additional information.

6.6.2.1 Fresh Concrete Tests. Concrete should be manufactured and transported to the job site in accordance with ASTM C94. Fresh concrete tests are summarized in Table 11.

Table 11
Fresh Concrete Tests

TEST DESCRIPTION	ASTM DESIGNATION
Making and curing laboratory concrete test specimens	C192
Making and curing field concrete test specimens	C31
Slump test	C143
Unit weight, yield, and air content (gravimetric)	C138
Air content of fresh concrete (volumetric method)	C173
Air content of fresh concrete (pressure method)	C231
Air content of fresh concrete (gravimetric method)	C138
Cement content	C138
Bleeding	C232
Time of setting by penetration	C403
Early volume changes	C827
Water retention by concrete curing materials	C156

6.6.2.2 Hardened Concrete Tests. Hardened concrete tests are summarized in Table 12.

Table 12
Hardened Concrete Tests

TEST DESCRIPTION	ASTM DESIGNATION
Air-void content	C457
Capping cylinders	C617
Compressive strength	C39
Accelerated curing and testing of concrete	C684
Splitting tensile strength	C496
Flexural strength	C78
Modulus of elasticity and Poisson's ratio	C469
Creep of concrete in compression	C512
Specific gravity, absorption and voids	C642
Cement content of hardened concrete	C85
Resistance to rapid freezing and thawing	C666
Abrasion resistance to sandblasting	C418
Abrasion resistance of horizontal surfaces	C779
Scaling resistance of concrete exposed to deicing salts	C672
Examining and sampling of concrete in constructions	C823

6.7 Mix Design Process.

6.7.1 Mix Design Procedures. Design concrete in accordance with ACI 211.1, Standard Practice for Selecting Proportions for Normal, Heavyweight and Mass Concrete, except as modified by guide specification NFGS-02520. All concrete shall be proportioned by weighing, except when equipment failures or other unusual circumstances necessitate the temporary use of volumetric proportioning. The minimum flexural strength at 28 days shall be 650 pounds per square inch (4.5 MPa).

6.7.2 Design of Lean Concrete Mixtures. Lean concrete mixtures described in Section 5 can be used as a base material. Mix procedures for normal weight concrete given in ACI 211.1 and NFGS-02520 can also be used for the design of lean concrete mixtures. Exceptions to the procedure include the use of a single aggregate instead of a combination of coarse and fine aggregates and a lower cement content than that used for normal weight concrete. The primary considerations in designing a lean concrete mixture are workability, adequate strength and durability. The mixture must be workable and easy to place and consolidate using vibration and standard concrete paving equipment. The mixture must also be cohesive enough to be placed with a slipform paver. Adequate strength and durability are required to withstand traffic loadings and environmental conditions.

Conduct laboratory testing of trial mixes to determine the cement content required to produce the desired workability and strength properties. Typical cement factors range from 200 to 350 pounds per cubic yard (119 to 208 kg per cubic meter) for lean concrete bases. Flexural strengths at 28 days should be between 300 and 400 pounds per square inch (2.1 and 2.8 MPa).

Air entrainment shall be used to improve freeze-thaw resistance and workability. Typical air contents for air-entrained lean concrete mixtures are between 3 and 9 percent. Maintain slumps in the 1- to 3-inch (25 to 76 mm) range when placing lean concrete mixtures. Specifications given in NFGS-02520 for concrete are also applicable for the mixing, transporting, and placement of lean concrete mixtures.

6.8 Hot Weather Considerations. The following sections provide general information on hot weather considerations.

6.8.1 Effects of High Temperatures. Hot weather is a combination of high air temperature, low relative humidity, and wind velocity which may impair the quality of fresh or hardened concrete. Hot weather can seriously affect the characteristics of plastic concrete. These include:

a) increased water demand,
b) earlier and more rapid slump loss,
c) increased rate of setting,
e) increased chance of plastic cracking,
e) problems in controlling entrained air content.

The hardened concrete properties can also be affected if additional water is added at the job site. These include:

a) decreased strength,
b) decreased durability (particularly at the surface),
c) increased chance of shrinkage and thermal cracking.

Hot weather can also affect transportation, placement, and finishing procedures by shortening the amount of time for each task. Do not add additional water at the site to offset the effects of high temperatures because of the corresponding decreased strength, durability of the surface, and impermeability, as well as the increased shrinkage cracking that may result from the higher water-cement ratio. A retarding admixture meeting the requirements of ASTM C494 could be beneficial in offsetting the accelerating effects of high temperatures and providing a uniform setting time.

6.8.2 Cooling Concrete Materials. Methods to maintain low concrete temperatures in hot weather include cooling aggregates and cooling water before mixing. The concrete temperature can be lowered by cooling the water, even though the concrete temperature is primarily controlled by the aggregate temperature. The engineer can use the following formula to calculate the approximate concrete temperature:

EQUATION:

$$T = \frac{0.22(T_a W_a + T_c W_c) + T_w W_w + T_{wa} W_{wa}}{0.22(W_a + W_c) + W_w + W_{wa}} \qquad (1)$$

WHERE:

- T = temperature of freshly mixed concrete, deg. F
- T_a = temperature of aggregates, deg. F
- T_c = temperature of cement, deg. F
- T_w = temperature of mixing water, deg. F
- T_{wa} = temperature of free moisture on aggregates, deg. F
- W_a = weight of aggregates, pounds
- W_c = weight of cement, pounds
- W_w = weight of mixing water, pounds
- W_{wa} = weight of free moisture on aggregates, pounds

Source: Portland Cement Association, Design and Control of Concrete Mixtures, Twelfth Edition.

If ice is used to aid in cooling, the concrete temperature can be approximated as follows:

EQUATION:

$$T = \frac{0.22(T_a W_a + T_c W_c) + (W_w - W_i)T_w}{0.22(W_a + W_c) + W_w + W_{wa} + W_i} + \frac{W_{wa} T_a - 112 W_i}{0.22(W_a + W_c) + W_w + W_{wa} + W_i} \qquad (2)$$

WHERE: W_i = weight of ice, pounds

Source: Portland Cement Association, Design and Control of Concrete Mixtures, Twelfth Edition.

The water temperature can be controlled with liquid nitrogen, the addition of ice, or insulating water supply lines and tanks. Aggregates can be cooled by shading stockpiles from the sun. The temperature of the concrete should not exceed 95 deg. F (35 deg. C) when the outdoor ambient temperature is more than 90 deg. F (32 deg. C). Additional information can be found in ACI 305, Hot Weather Concreting, and NFGS-02559.

6.8.3 Transporting, Placing, and Finishing. Transporting and placing concrete during hot weather should be completed as quickly as possible to prevent loss of workability and cold joints. Ready mix concrete must meet ASTM C94 which requires discharge of concrete within 1.5 hours or before 300 revolutions of the drum, whichever comes first. This time limit could be reduced to 45 minutes for ambient temperatures above 90 deg. F (32 deg. C). Coordinate deliveries to avoid delays at the construction site. Concrete may also be placed at night to take advantage of cooler temperatures during hot weather periods. See NFGS-02559 for specifications on mixing, placing, and finishing.

6.8.4 Plastic Cracking. Plastic cracking occurs when moisture from the fresh concrete surface evaporates rapidly. This rapid loss of water results in shrinkage and tensile stresses in the concrete which lead to plastic cracks. High air and concrete temperatures, low humidity, and high winds all increase the possibility of plastic cracking. The following precautions can minimize plastic cracking and should be considered during hot weather:

a) moistening subgrade and forms,
n) moistening dry and absorptive aggregates,
c) lowering fresh concrete temperatures by cooling aggregates and mixing water,
d) protecting concrete between placing and finishing with temporary coverings,
e) reducing the time between placing and curing,
f) moist curing.

6.8.5 Curing. The goal of curing is to maintain sufficient water in the concrete to continue hydration and maximize strength gain. To obtain proper curing during hot weather it is important to reduce moisture loss and prevent plastic cracking. Strength gain will stop when sufficient moisture for curing is no longer available. Continuous moist curing is preferred. If moist curing cannot be accomplished, or if it cannot be continued beyond 24 hours, protect the concrete surface for the remainder of the curing period with plastic sheets, membranes, etc. See NFGS-02520 for specifications on curing materials.

6.9 Cold Weather Considerations. The following sections provide general information on cold weather considerations. See ACI 306, Cold Weather Concreting, for additional information on cold weather concreting practices.

6.9.1 Effects of Cold Weather. Reduced temperatures slow down the rate of hydration and the associated gain in strength. To reduce the chance of damaging concrete by early freezing, use air-entrained concrete in cold weather construction. Air-entrained concrete protects the concrete by providing a reservoir for water that is forced from the cement paste during freezing. High early strength gain is desirable in cold weather operations to reduce the length of time that temporary protection is required. To facilitate early strength gain, use a high-early-strength cement (Type III), an insulation layer, or an accelerating admixture.

6.9.2 Heating of Concrete Materials. It is desirable to prepare fresh concrete to a temperature in the range of 50 deg.F (10 deg. C) to 90 deg. F (32 deg.C) for cold weather construction. Methods to maintain concrete temperatures in cold weather include heating aggregates and water before mixing. Aggregates can be heated by circulating steampipes through stockpiles and by covering stockpiles to retain heat. Use Equation (1) to calculate the approximate concrete temperature.

Do not place concrete when the air temperature in the shade falls below 40 deg. F (4 deg. C), or when the concrete, without special protection, is likely to be exposed to freezing temperatures prior to the completion of the designated curing period. See NFGS-02520 for specifications on heating mixing water and aggregates.

6.9.3 Transporting, Placing, and Finishing. During cold weather, place concrete before the mix temperature drops below a temperature range of 60 deg. F (16 deg. C) to 70 deg. F (21 deg. C). See NFGS-02520 for specifications on placing concrete in cold weather.

6.9.4 Curing. Concrete must be protected from rapid temperature drop when curing. Effective means of insulation include straw or hay and commercial insulation blankets. See NFGS-02520 for specifications on curing materials.

PAGE 44 INTENTIONALLY BLANK

Section 7: AIRCRAFT TRAFFIC

7.1 General. An airfield pavement must be designed to support a forecast number of loadings by one or more types of aircraft expected to use the facility over a selected design period. This requires information about:

a) the aircraft types (gear configurations),
b) the maximum gross weight of each aircraft type,
c) the lateral wander associated with each aircraft type,
d) the predicted number of operations of each aircraft type over the design life of the pavement.

All of these items are input into the pavement thickness design procedure provided in Section 8.

7.2 Aircraft Types. Most Navy and Marine Corps aircraft have a tricycle landing gear. A landing gear assembly may consist of single wheel for smaller aircraft or dual and dual tandem wheels for larger aircraft. Figure 3 illustrates the various multiwheel landing gear assemblies and lists typical aircraft for each.

One aircraft in each gear assembly group has been designated the representative aircraft for that group. Table 13 identifies these five standard aircraft types which are to be used as default values in the design of rigid pavements only when site-specific aircraft loadings are not available. Table 14 provides the design gear loads and tire pressures for most other current aircraft.

Table 13
Standard Design Aircraft Types

LANDING GEAR ASSEMBLY	REPRESENTATIVE AIRCRAFT	WHEEL SPACING INCHES	(mm)	TIRE PRESSURE PSI	(MPa)	DESIGN GEAR LOAD POUNDS	(kg)
Single	F-14	-	-	240	(1.65)	30,000	(13 600)
Dual	P-3	26	(660)	190	(1.31)	68,000	(30 800)
Single Tandem	C-130	60	(1524)	95	(0.65)	84,000	(38 100)
Dual Tandem	C-141	32.5x48	(826x1219)	180	(1.24)	155,000	(70 300)
Twin Delta Tandem	C-5A	FIGURE 8		115	(0.79)	190,000	(86 200)

7.3 Design Weight. Use the design gear loads given in Table 14 for pavement thickness design. The design gear load is calculated from the design gross aircraft weight (typically the maximum gross take-off weight) by assuming that 95 percent of the gross aircraft weight is carried by the main gears. The design gear loads given in Tables 13 and 14 were calculated from the maximum gross take-off weights of the listed aircraft, and thus represent the maximum static gear loads expected to be applied to a pavement.

7.3.1 Use of Other Gear Loads in Design. Other gear loads than those listed in Tables 13 and 14 may be used for design when warranted. Certain areas of an airfield (e.g., runway shoulders, runway overruns) do not normally carry fully loaded aircraft and thus do not have to be designed for the maximum gross weight. The pavement thickness design charts in Section 8 accommodate a wide range of gear loads.

7.3.2 Hangar Floors. Aircraft in hangars are not normally loaded with cargo, fuel or armaments. Design hangar floors for the empty weight of the aircraft. When exact data are not available, use 60 percent of the maximum gross weight of the aircraft as an average. The minimum thickness of the concrete slab in a hangar floor is 8 inches (203 mm).

7.4 Traffic Areas. Airfield pavements are categorized by traffic area as a function of traffic distribution and aircraft weight. The three principal traffic areas recognized on Navy and Marine Corps air stations are primary, secondary, and supporting. For purposes of standardization and for preparation of this Tri-Service design criteria, a primary area corresponds to an Air Force B traffic area and secondary traffic areas correspond to an Air Force C traffic area.

7.4.1 Primary Traffic Areas. Primary traffic areas are those which require high pavement strength due to the combination of high operating weights and channelized traffic. Primary traffic areas include:

a) first 1000 feet (330 m) of runways,
b) primary taxiways,
c) holding areas,
d) aprons.

7.4.2 Secondary Traffic Areas. Secondary traffic areas are normally subjected to unchannelized traffic and/or aircraft operating at lower weights than primary traffic areas. Secondary traffic areas include:

a) runway interiors,
b) intermediate taxiway turnoffs.

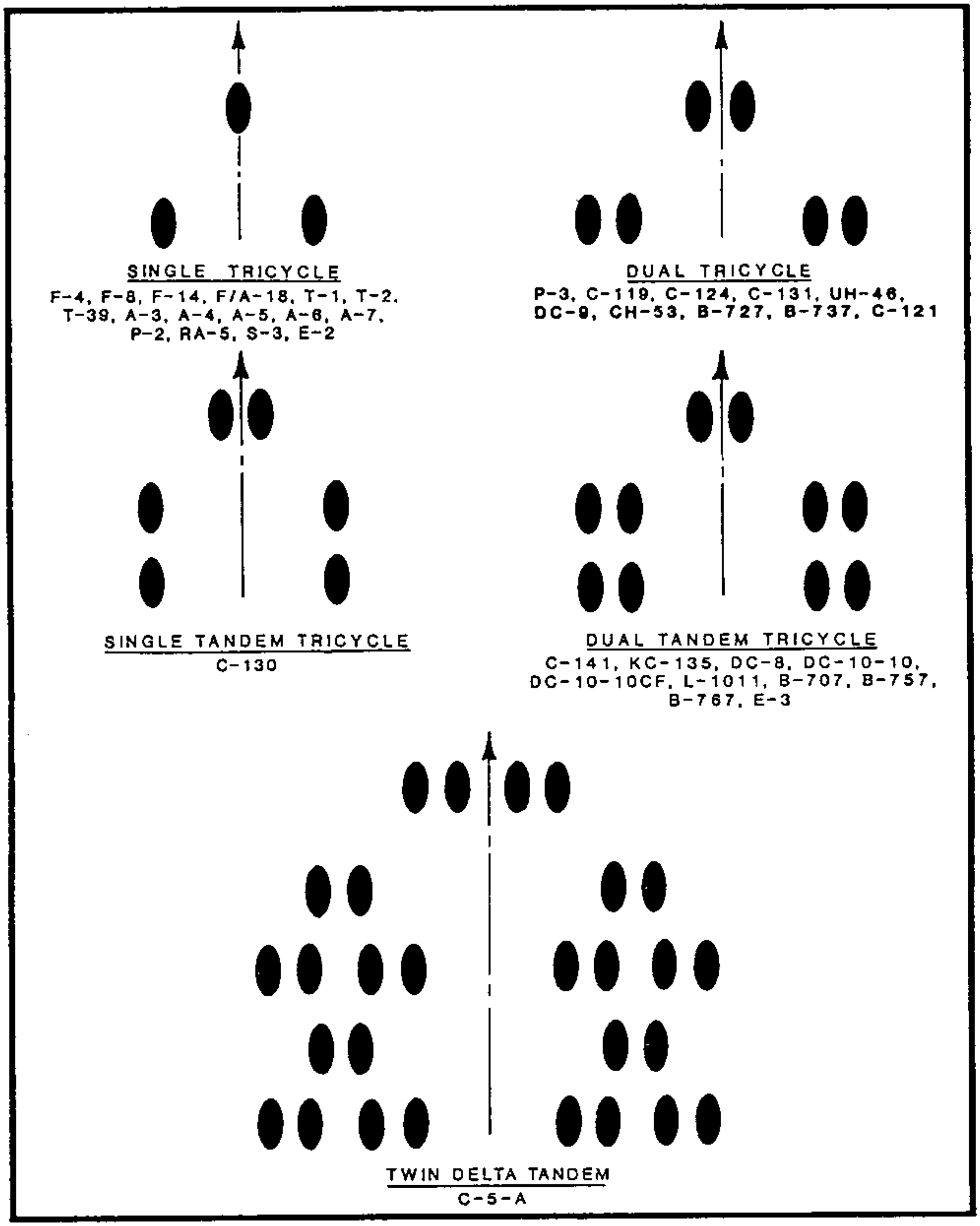


Figure 3
Multiwheel Landing Gear Assemblies

Table 14
Commonly Used Design Loadings for Navy and Marine Corps Air Stations

Type	DOD Designation	Type of Landing Gear**	Design Gear Load	Design Tire Pressure	Design Figures	Pass/Coverage Chan.	Pass/Coverage Unchan.
Attack	A-3	ST	37,000 lb,	245 psi	4,9	3.48	14.96
	A-4	ST	12,500 lb,	200 psi	4,9	11.63	23.26
	A-5	ST	29,500 lb,	300 psi	4,9	9.27	18.54
	RA-5	ST	38,000 lb,	350 psi	4,9	8.82	17.64
	A-6	ST	28,700 lb,	200 psi	4,9	7.67	15.35
	A-7	ST	21,000 lb,	200 psi	4,9	8.97	13.91
	AV-8	DT	14,000 lb,	90 psi	-	3.89	7.47
Fighter	F-4	ST	22,500 lb,	500 psi	4,9	13.70	27.39
	F-8	ST	18,000 lb,	400 psi	4,9	13.69	27.39
	F-14	ST	30,000 lb,	240 psi	4,9	8.58	17.00
	F/A-18*	ST	25,000 lb,	200 psi	4,9	8.22	16.44
Trainer	T-1	ST	9,000 lb,	200 psi	4,9	13.69	27.39
	T-2	ST	7,000 lb,	165 psi	4,9	14.10	28.20
	T-39	ST	9,000 lb,	165 psi	4,9	12.45	24.89
Patrol	P-3	DT	68,000 lb,	190 psi	5,10	3.45	6.49
	S-3	ST	19,000 lb,	245 psi	4,9	10.43	20.87
Transport and Tanker	C-5A	TDT	190,000 lb,	115 psi	8,13	1.62	2.20
	C-121	DT	81,000 lb,	170 psi	5,10	3.45	6.18
	C-130	STT	84,000 lb,	95 psi	6,11	4.36	8.56
	KC-135	DTT	142,000 lb,	155 psi	7,12	3.37	5.97
	C-141	DTT	155,000 lb,	180 psi	7,12	3.49	6.25
Rotary Wing	UH-46	DT	9,800 lb,	150 psi	--	-	-
	CH-53H	DT	26,000 lb,	165 psi	--	-	-
Commercial	B-707	DTT	157,000 lb,	180 psi	7,12	3.30	5.87
	B-727	DT	98,000 lb,	150 psi	5,10	3.30	5.88
	B-737	DT	54,000 lb,	150 psi	5,10	3.20	5.80
	B-747	DTT	190,000 lb,	195 psi	7,12	3.84	5.43
	B-757	DTT	105,000 lb,	170 psi	7,12	3.30	5.88
	B-767	DTT	143,000 lb,	183 psi	7,12	3.71	6.05
	DC-8	DTT	172,000 lb,	196 psi	7,12	3.19	5.82
	DC-9	DT	57,000 lb,	170 psi	5,10	3.61	6.73
	DC-10	DTT	210,500 lb,	165 psi	7,12	3.77	5.61
	(Center Dual)		91,100 lb,	140 psi	5,10	2.63	3.96
	L-1011	DTT	219,000 lb,	165 psi	7,12	3.66	5.57
Early Warning	E-2	ST	24,500 lb,	260 psi	4,9	8.58	17.00
	E-3	DTT	155,000 lb,	180 psi	7,12	3.30	5.87

* The nose gear on the F/A-18 carries 20 percent of the aircraft load.

** ST = Single Tricycle
DT = Dual Tricycle
TDT = Twin Delta Tandem
STT = Single Tandem Tricycle
DTT = Dual Tandem Tricycle

7.4.3 Supporting Areas. Supporting areas are not intended for normal aircraft operations. They are designed to withstand occasional passes of aircraft on an emergency basis. Supporting traffic areas include:

a) inner 10 feet (3.3 m) of runway shoulders,
b) stabilized portions of runway overruns,
c) blast protective pavement.

7.5 Pass-Coverage Ratio. In order to accurately quantify the fatigue damage that aircraft loadings will do to a pavement over its design life, consider the lateral distribution of the aircraft as well as its wheel/gear configuration. The forecasted number of aircraft passes over a pavement are converted to coverages. Coverages are defined as the maximum number of tire prints or partial tire prints applied to the pavement surface at that point where maximum accumulation occurs (i.e., in the center of the wheel path).

Coverages resulting from operations of a particular aircraft type are a function of the number of passes, the number and spacing of wheels on the main landing gear, the width of the tire contact area, and the lateral distribution of the aircraft wheel paths relative to the pavement centerline. The collective influence of these factors is expressed in terms of pass-coverage ratios derived for each aircraft. Tables 14 and 15 give the pass-coverage ratios to be used for aircraft for primary (channelized) and secondary (unchannelized) traffic areas.

Table 15
Pass-Coverage Ratios for Standard Aircraft

Aircraft	Channelized P/C*	Unchannelized P/C**
F-14	8.58	17.00
P-3	3.45	6.49
C-130	4.36	8.56
C-141	3.49	6.25
C-5A	1.62	2.20

* Computed using a normal distribution and standard deviation of 30.43 in.
**Computed using a normal distribution and standard deviation of 60.87 in.

References: "Lateral Distribution of Aircraft Traffic," D.N. Brown and O.O. Thompson, Miscellaneous Paper S-73-56, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, July 1973.

"Field Survey and Analysis of Aircraft Distribution on Airport Pavements" V.A. HoSang, Report No. FAA-RD-74-36, Federal Aviation Administration, Washington, D.C., February 1975.

7.6 Traffic Volume. Determine the total number of passes of each aircraft type that the pavement will be expected to support over its design life. The minimum design life for Navy and Marine Corps facilities is 20 years. Normally only aircraft departures are included as passes in pavement thickness design. The exception to this is in touchdown areas on runways where the impact due to aircraft performing touch-and-go operations will cause pavement damage. On pavements that are to be used for touch-and-go operations add the expected number of touch-and-go operations over the design life to the number of departures to arrive at the design traffic. Obtain data for the specific Navy and Marine Corps airfield facility under design to forecast aircraft traffic operations over the design life of the pavement.

When site-specific traffic projections are not available, the aircraft pass levels listed in Table 16 are the minimum pass levels to be used in design.

Table 16
Minimum Pass Levels for Standard Aircraft

Aircraft	Total Passes Over Design Life*
F-14	300,000
P-3	100,000
c-130	50,000
c-141	25,000
C-5A	25,000

* Departures at maximum gross load.

Section 8: THICKNESS DESIGN FOR NON-REINFORCED CONCRETE PAVEMENT

8.1 Basis for Design. The thickness design procedure is based upon providing a sufficient structural capacity of the pavement system for a specified mix of aircraft loadings. The key structural design factors include:

1) slab thickness,
2) slab concrete flexural strength,
3) foundation support (from subgrade and base),
4) aircraft types, weights, passes and pass-coverage ratios.

It is also assumed that a relatively short joint spacing and adequate load transfer at the joints are provided.

8.1.1 Fatigue Damage. Repeated aircraft loading results in fatigue damage in the concrete slabs which results in microcracks at the bottom of the slab. These cracks work their way to the surface of the slab, eventually dividing the slab into two or more pieces. In addition, if pumping and loss of support occur at slab corners, the critical stress could increase until a corner break develops. As the proportion of cracked slabs increases, the airfield pavement requires increasing maintenance and repair.

8.1.2 Structural Characterization. The slab and foundation are characterized using the Westergaard theory (Westergaard, H.H., New Formulas for Stresses in Concrete Pavements of Airfields) of a slab loaded at the interior resting on a uniformly supported foundation (as modeled using the k value). All stresses are computed using the computer program PCAPAVE developed by the Portland Cement Association (Packard, R.G., Computer Program for Airfield Pavement Design) for interior stress condition. A major design assumption is that adequate load transfer is provided at the joints so that the load stresses that occur at the joints are not significantly higher than the stresses at the interior of the slab. Adequate load transfer must be provided by a stabilized base, keyways, mechanical load transfer devices or aggregate interlock.

8.1.3 Structural Slab Cracking from Aircraft Loadings. The cracking of a non-reinforced jointed concrete slab with relatively short joint spacing is controlled by:

a) the magnitude of flexural stress caused by aircraft traffic,
b) the flexural strength of the concrete,
c) the number of stress applications.

The number of allowable stress applications to crack the concrete slab is controlled by the ratio of critical stress to flexural strength of the concrete. The relationship used in this design procedure to relate stress/flexural strength ratio to the number of stress applications to cracking was developed by the PCA and is shown in Table 17. The lower the ratio of critical stress to flexural strength, the larger the number of load applications that the slab can carry before cracking occurs.

Table 17
Stress-Strength Ratios and Allowable Coverages

Stress-Strength* Ratio	Allowable Coverages	Stress-Strength* Ratio	Allowable Coverages
0.45	2,300,OOO	0.63	14,000
0.46	1,700,000	0.64	11,000
0.47	1,300,000	0.65	8,000
0.48	1,000,000	0.66	6,000
0.49	720,000	0.67	4,500
0.50	540,000	0.68	3,500
0.51	400,000	0.69	2,500
0.52	300,000	0.70	2,000
0.53	240,000	0.71	1,500
0.54	180,000	0.72	1,100
0.55	130,000	0.73	850
0.56	100,000	0.74	650
0.57	75,000	0.75	480
0.58	57,000	0.76	370
0.59	42,000	0.77	280
0.60	32,000	0.78	210
0.61	24,000	0.79	160
0.62	18,000	0.80	120

* Interior or edge stress and design flexural strength

8.4.1 Structural Slab Cracking and Mixed Aircraft Loading. When two or more aircraft will utilize a given pavement, each may cause a certain amount of fatigue damage in the concrete slab. The effect of mixed traffic can be provided for in the pavement design by using Miner's cumulative fatigue damage procedure as described in Cumulative Damage in Fatigue, H.A. Miner. Fatigue damage is defined as the ratio of the number of loading cycles actually applied (at a given stress level) to the number of allowable load applications to cracking failure (at the same stress level). The resulting fraction represents the proportion of the useful life of the concrete that is consumed by repeated loading.

EQUATION: Fatigue Damage = $\sum n_i / N_i$ (3)

WHERE:
n_i = number of applied loads (coverages) at a given stress level (as denoted by i)
N_i = number of allowable loads (coverages) at the same stress level to cracking of the concrete

The fatigue damage can be accumulated over any number of stress levels (or different aircraft loadings) as indicated by the summation sign. When the fatigue damage proportion reaches 1.0, or 100 percent, a substantial number of slabs will have cracked.

8.2 Thickness Design Inputs. Five key design inputs are needed to determine the required slab thickness.

8.2.1 Design Concrete Flexural Strength. The 28-day third-point loading flexural strength is used for pavement design. The design flexural strength should be as high as practicable and economical but not less than 650 pounds per square inch (4 482 000 Pa). The actual mean flexural strength in the field will be greater than the design flexural strength. The concrete design shall meet the flexural strength requirements in NFGS-02520.

8.2.2 k Value at Top of Base. The determination of the k value on the subgrade and at the top of the base layers is described in Section 5. The value used for design is that obtained at the top of the base. The combined base and subgrade should have a minimum design k value of 200 pounds per cubic inch (5 536 000 kg per cubic meter) to prevent excessive permanent deformation of the subgrade due to slab corner deflections. A base course of sufficient thickness and quality should be used to achieve this modulus. However, in no case should design be based on a k value greater than 500 pounds per cubic inch (13 840 000 kg per cubic meter).

A stabilized base or lean concrete base may be used as a substitute for a granular base course on a 1:1.5 thickness replacement ratio. However, the k value used for design remains the same as that determined at the top of the granular base. The design k value is not increased due to the use of a stabilized or lean concrete base. An unbonded stabilized or lean concrete base does not increase the effective k value greatly due to slippage between the slab and base.

8.2.3 Type and Design Gear Load of Aircraft Using Facility. Pavement thickness design can be determined for a single design aircraft or for a mix of aircraft traffic. Determine the design gear load for a given aircraft by first selecting the design gross aircraft weight. This is normally the maximum gross aircraft weight at departure. Then estimate the design gear load by assuming that 95 percent of the gross weight is carried by the main gears. Design values are given in Tables 13 and 14 in Section 7.

8.2.4 Number of Aircraft Passes. Forecast the total number of passes (not coverages) of each aircraft that is expected to use the pavement feature over its design life. Some general recommendations are provided in Section 7. The "number of passes" is normally the number of departures at the maximum gross aircraft weight. The exception to this is in touchdown areas on runways where the impact due to aircraft performing touch-and-go operations will cause

8.2.5 Primary or Secondary Traffic Area. Guidance on determining if the pavement feature is a primary or secondary traffic area is given in Section 7

8.3 Thickness Design Procedure for a Single Design Aircraft. Use Figures 4 through 8 to determine the concrete slab thickness for single design aircraft. These charts were developed using stresses that were computed using the computer program PCAPAVE developed by the PCA for the interior stress condition.

The design chart for aircraft with single wheel gear is shown in Figure 4 and is used by entering the design flexural strength and the tire load and projecting as shown by the dashed example lines until the required slab thickness is obtained. The design charts shown in Figures 5 through 8 are also entered with the design concrete flexural strength and projecting as shown by the dashed example lines until the required slab thickness is obtained.

The calculated slab thickness is rounded to a whole inch to obtain the design thickness. If the computed thickness is less than or equal to 0.25 inches (6 mm> greater than a whole inch, the thickness is rounded downward (e.g., 10.15 inches (256 mm) is rounded to 10 inches (254 mm)>. If the thickness is more than 0.25 inches (6 mm) greater than a whole inch, the thickness is rounded upward (e.g. 10.30 inches is rounded to 11 inches (279 mm>).

This procedure will provide the required slab thickness for a specified type of aircraft when the flexural strength, k value, gear load, tire pressure for single wheel gear aircraft, number of passes and type of design traffic area are specified. The example presented in Table 18 illustrates how to determine the required slab thickness using the design chart for a single design aircraft.

If design charts are needed for any aircraft not included in Figures 4 through 8, use the PCA computer program PCAPAVE through the NAVFAC Engineering and Design Division Time Sharing Computer Library along with the fatigue life data given in Table 17 to develop a required pavement design.

Table 18
Design Example for a Single Design Aircraft

Aircraft = C-141
Design gear load = 155,000 pounds (70 300 kg)
Design flexural strength = 650 pounds per square inch (4.5 MPa)
Effective k value at top of base course = 200 pounds per cubic inch ($5\ 536\ 000\ kg/m^3$)
Total departures over 20-year design life = 25,000
Traffic area = primary taxiway (channelized traffic)
Required slab thickness = 13.4 inches (340 mm) (from Figure 7)
This thickness should be rounded up to 14.0 inches (356 mm)

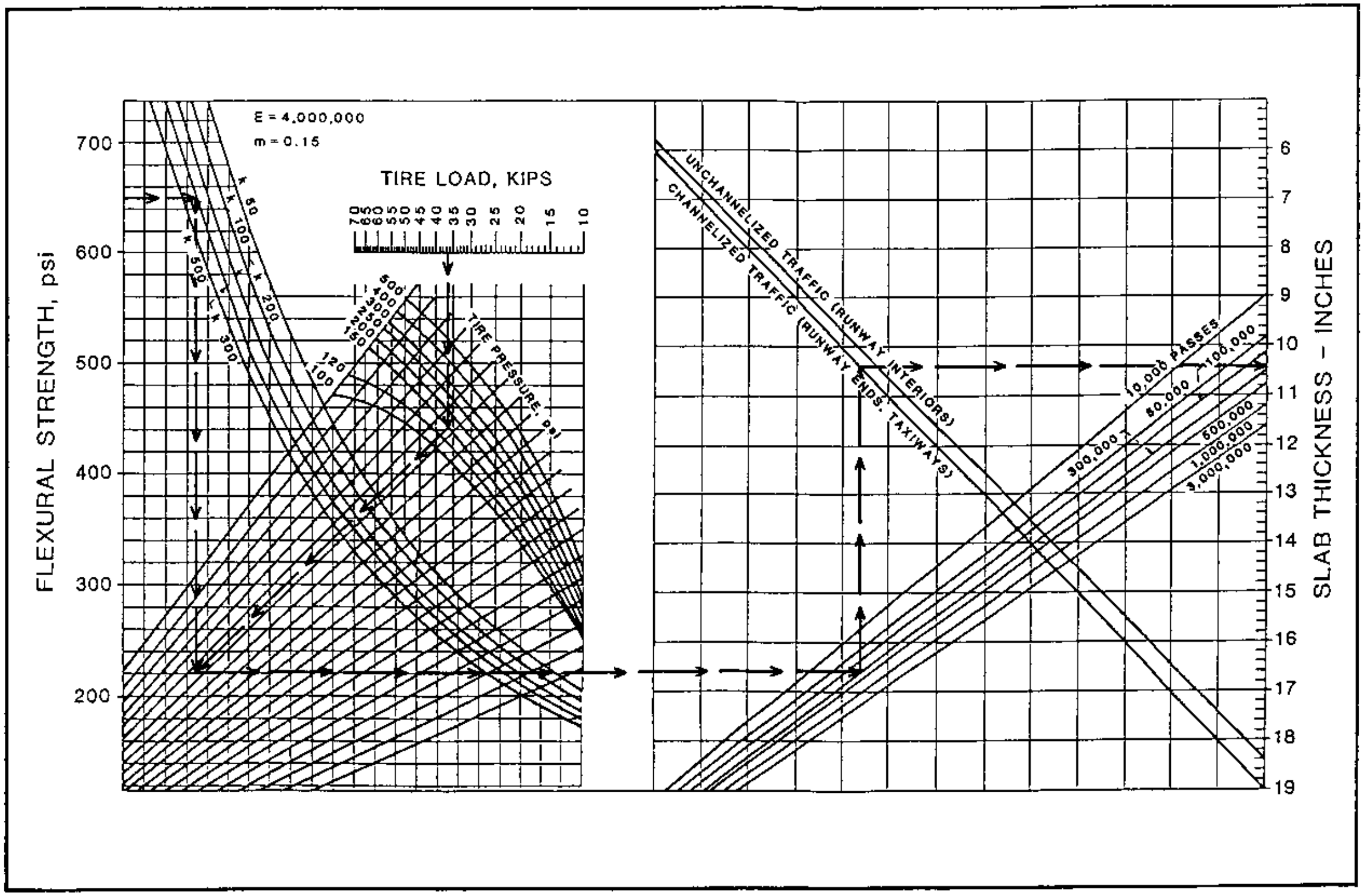


Figure 4
Rigid Pavement Thickness Design Chart for Single Wheel Gear

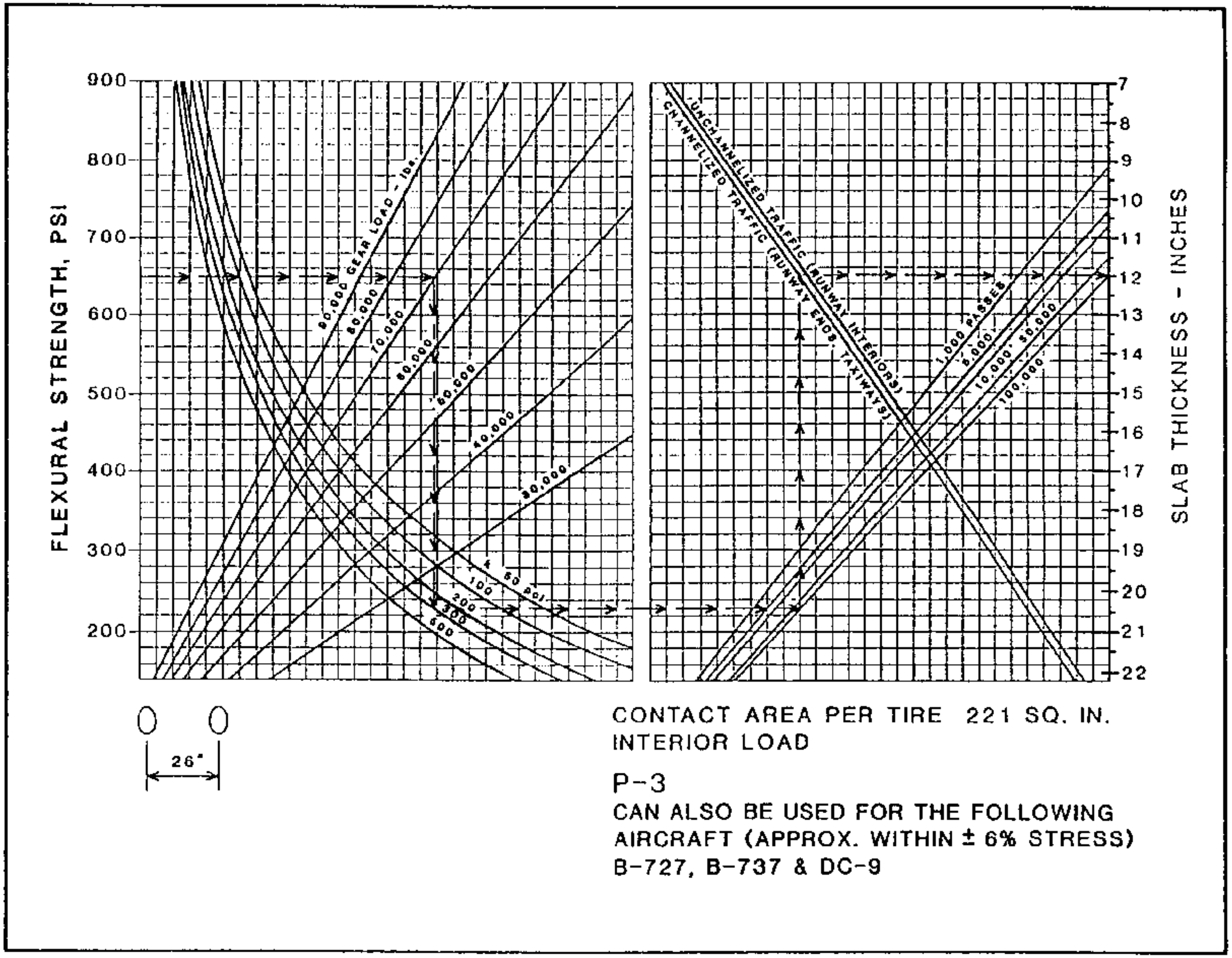


Figure 5
Rigid Pavement Thickness Design Chart for P-3 Aircraft

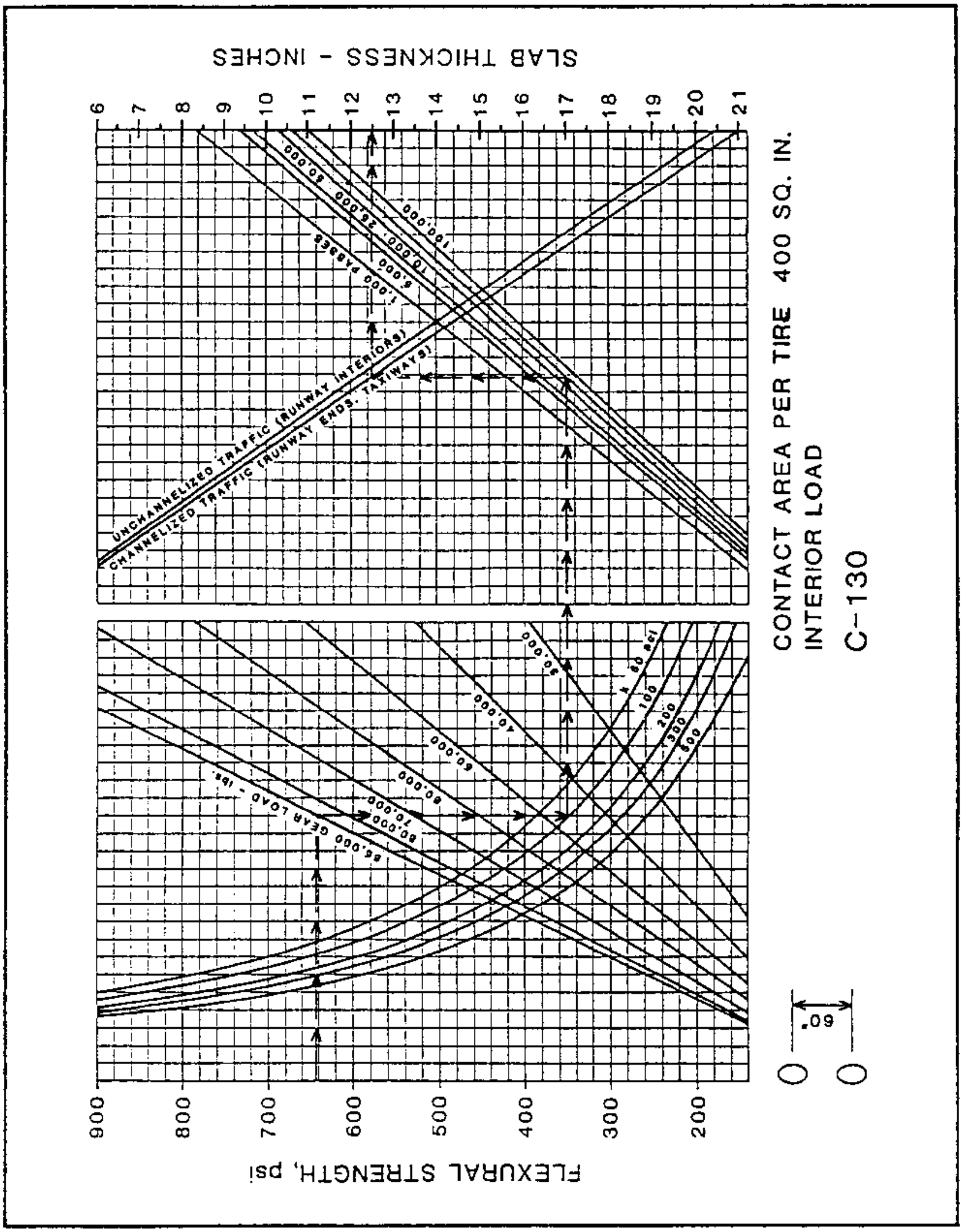


Figure 6
Rigid Pavement Thickness Design Chart for C-130 Aircraft

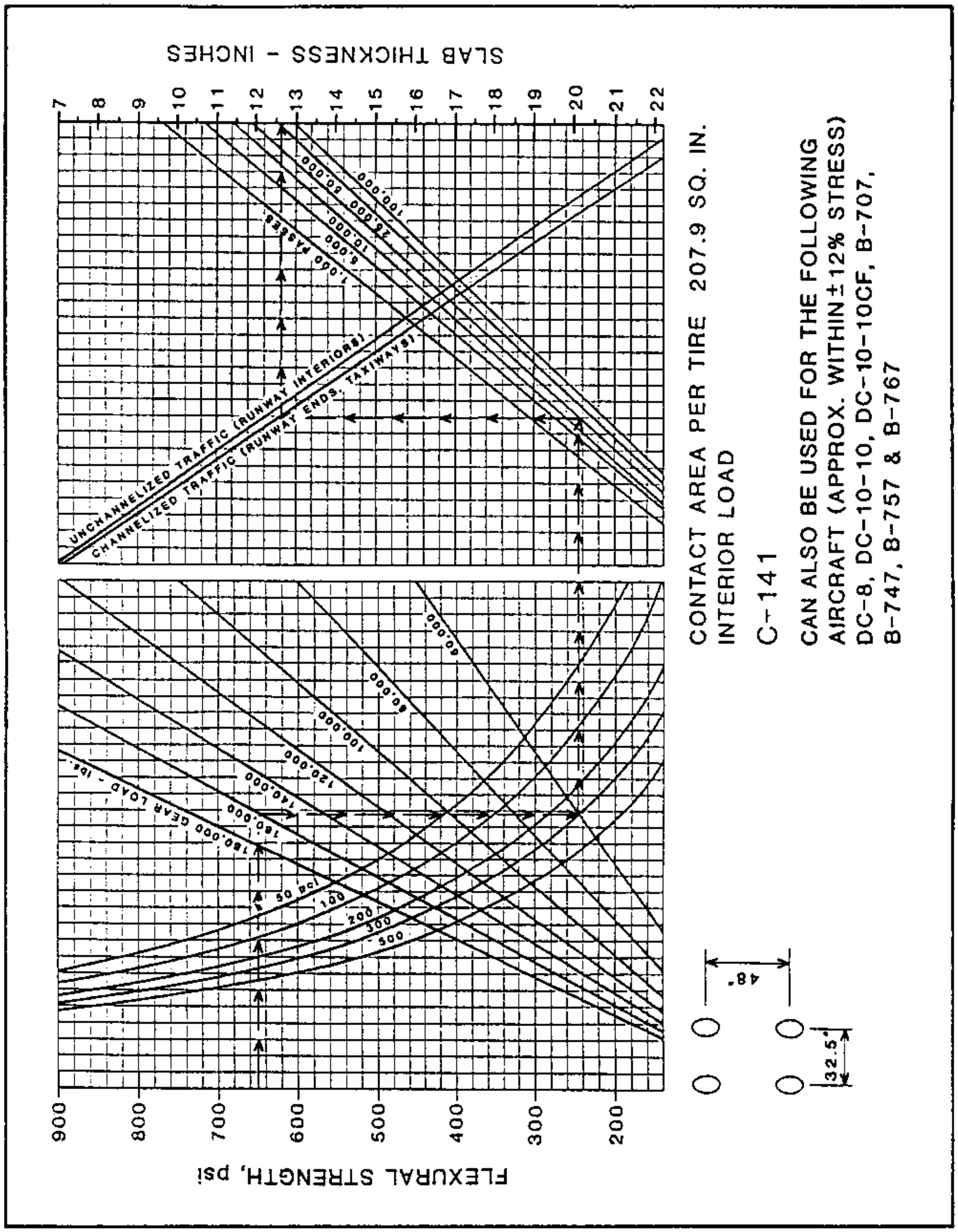


Figure 7
Rigid Pavement Thickness Design Chart for C-141 Aircraft

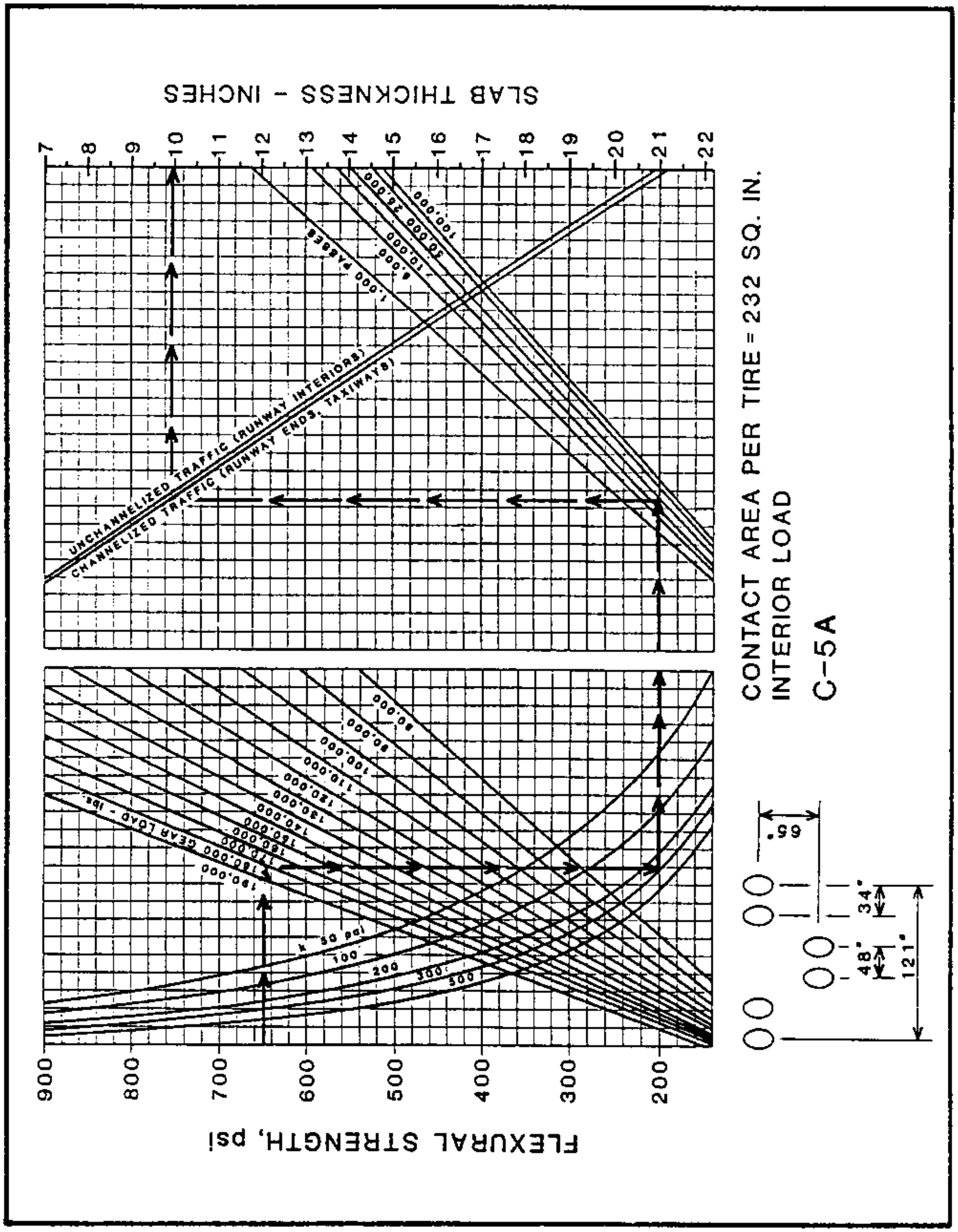


Figure 8
Rigid Pavement Thickness Design Chart for C-5A Aircraft

8.4. Thickness Design Procedure for Mixed Traffic. When an airfield pavement will be loaded by two or more aircraft, the combined damage caused by the aircraft mix must be used in the design.

The required slab thickness may be determined for a mix of aircraft types using Miner's damage hypothesis and data on forecasted operations of different aircraft types operating at the facility. The slab thickness design for mixed traffic is an iterative procedure in which the designer selects a trial slab thickness (normally the thickness required for the most critical aircraft using the feature plus one inch), computes the proportion of the fatigue life of the pavement consumed as the sum of the individual damage contributions of the forecasted volume of each aircraft type, and subsequently varies the slab thickness until less than 100 percent of the fatigue life of the pavement is consumed by the forecasted mix of traffic.

This procedure is described in the following sections and is facilitated by the use of a table for computations as shown in Table 19. Photocopies of Table 19 should be made for the designer's use.

8.4.1 Required Inputs. The specific aircraft types and their design gear load (typically 95 percent of the maximum gross departure gear load) are entered in Columns 1 and 2 of Table 19. The projected number of passes (departures) over the selected design period are entered in Column 3 of Table 19. Divide the projected passes by the appropriate pass-coverage ratio from Table 15 to obtain projected coverages for each aircraft. If the forecasted number of passes is not available, use the minimum pass levels given in Table 16.

Use the pass-coverage ratios given for primary (channelized) traffic areas when designing for runway ends, primary taxiways, and aprons. Use the pass-coverage ratios given for secondary (unchannelized) traffic areas when designing for other areas, Enter the pass-coverage ratio selected for each aircraft in Column 4 of Table 19, and the number of coverages computed in Column 5. The other required inputs are the concrete flexural strength, the effective k value on top of the base, and the tire pressure for each single wheel aircraft, which should be recorded in the spaces provided at the top of Table 19.

8.4.2 Determination of Interior Flexural Stresses. Select a trial slab thickness and record it in the space provided for the iteration being performed. For the initial trial, use the required thickness for the expected critical aircraft (determined from Figures 4 through 8) plus one inch.

Table 19
Fatigue Damage Summary Sheet for Mixed Traffic

PAVEMENT IDENTIFICATION ______________ TRAFFIC AREA ______________
SLAB THICKNESS: ______________ SINGLE WHEEL AIRCRAFT TIRE PRESSURE, psi
BASE K: ______________ 1. ______________ 1. ______________
FLEXURAL STRENGTH ______________ 2. ______________ 2. ______________

1 AIRCRAFT	2 DESIGN GEAR LOAD	3 PROJECTED PASSES	4 P/C	5 PROJECTED COVERAGES (n)	6 INTERIOR STRESS	7 STRESS/F S	8 ALLOWABLE COVERAGES (N)	9 FATIGUE LIFE CONSUMED (n/N)
							Σ n/N:	

Determine the flexural stress at the bottom of the slab, caused by each particular aircraft gear for the interior loading position, using Figures 9 through 13. For each single wheel gear aircraft, enter Figure 9 with the trial slab thickness, tire load, and tire pressure, move either up or down to the base effective k value and continue horizontally to the flexural stress. For each of the multiwheel gear aircraft types listed, enter the appropriate Figures 10 through 13 with the trial slab thickness, project a horizontal line left to the effective k value, move either up or down to the design gear load, and continue horizontally to the flexural stress. Record the stress values in Column 6 of Table 19.

8.4.3 Fatigue Life Consumption. The stress-strength ratio recorded for each aircraft in Column 7 of Table 19 is the flexural stress in Column 6 divided by the design concrete flexural strength. Select from Table 17 the allowable number of coverages corresponding to the stress-strength ratio computed for each aircraft type, and record the allowable number of coverages in Column 8 of Table 19. For each aircraft type, divide the projected number of coverages in Column 5 by the allowable number of coverages in Column 8 to determine the portion of fatigue life consumed by the forecasted volume of each aircraft type.

The sum of the values in Column 9 is Miner's damage, the proportion of total fatigue life of the slab consumed by the forecasted volumes of the aircraft types listed. If this number is considerably less than 1.00 (100 percent), indicating that the slab has considerable remaining fatigue life at the end of the design period not consumed by the forecasted mix of traffic, then the trial slab thickness may be reduced in the next iteration.

If the Miner's damage is greater than 1.00 (100 percent), indicating that the fatigue life of the slab will be consumed by lower traffic volumes than those projected over the design period, then the trial slab thickness must be increased in the next iteration.

The process of selecting a slab thickness, determining the flexural stress, and calculating the fatigue life consumption is repeated until the slab thickness which corresponds to an acceptable value for Miner's damage (less than 1.00 or 100 percent) is determined.

The calculated slab thickness is rounded to the nearest whole inch to obtain the design thickness. If the computed thickness is less than or equal to 0.25 inches (6 mm) greater than a whole inch, the thickness is rounded downward (e.g., 10.15 inches (256 mm) is rounded to 10 inches (254 mm)). If the thickness is more than 0.25 inches (6 mm) greater than a whole inch, the thickness is rounded upward (e.g., 10.30 inches is rounded to 11 inches (279 mm)).

8.5 Minimum Thickness. The minimum allowable new concrete pavement thickness is 8 inches (203 mm) in primary and secondary traffic areas and 4 inches (102 mm) in blast protective areas not subject to aircraft loading. For helicopter and basic training fields the minimum thickness in primary and secondary traffic areas is 6 inches.

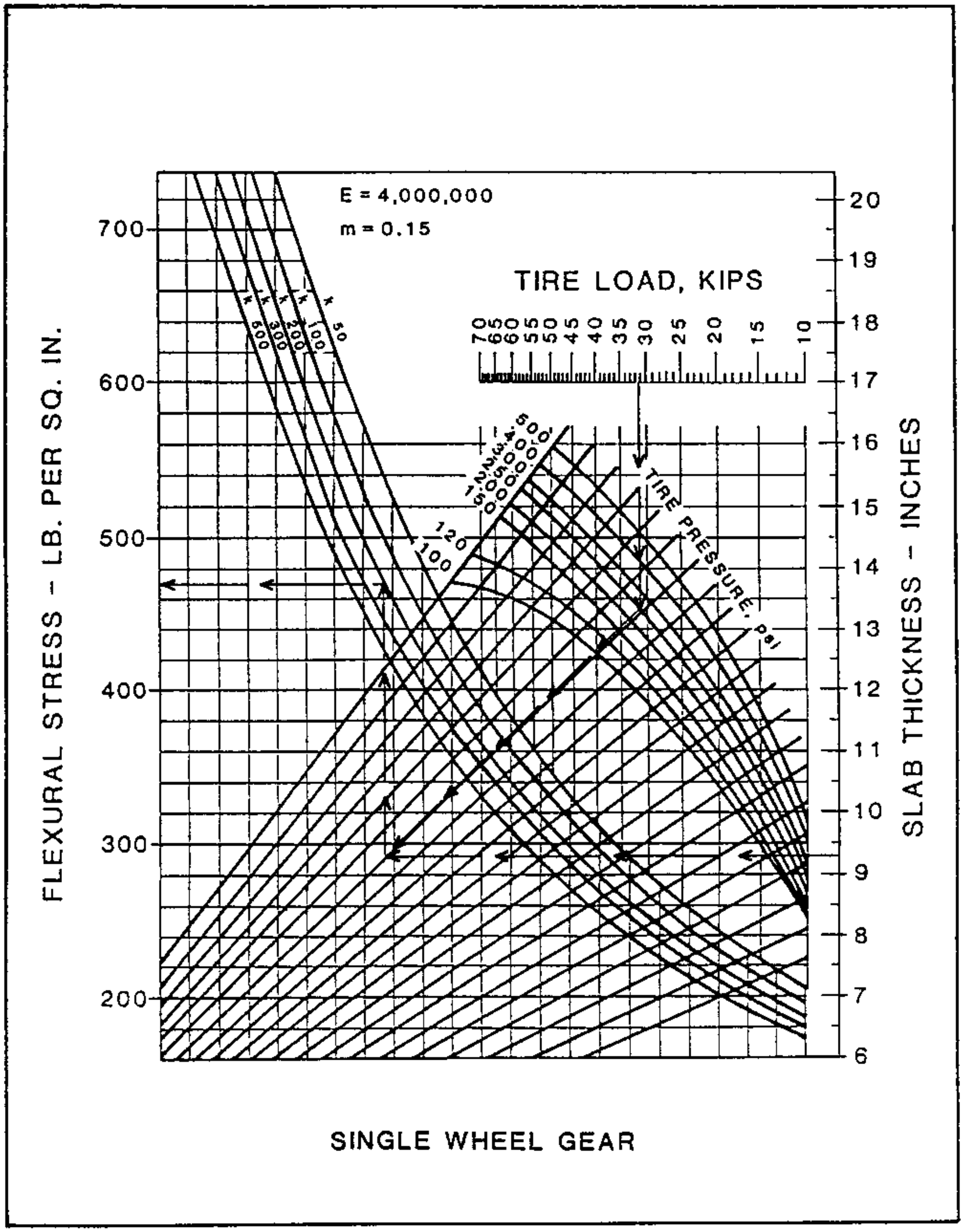


Figure 9
Chart for Determining Flexural Stress for Single Wheel Gear

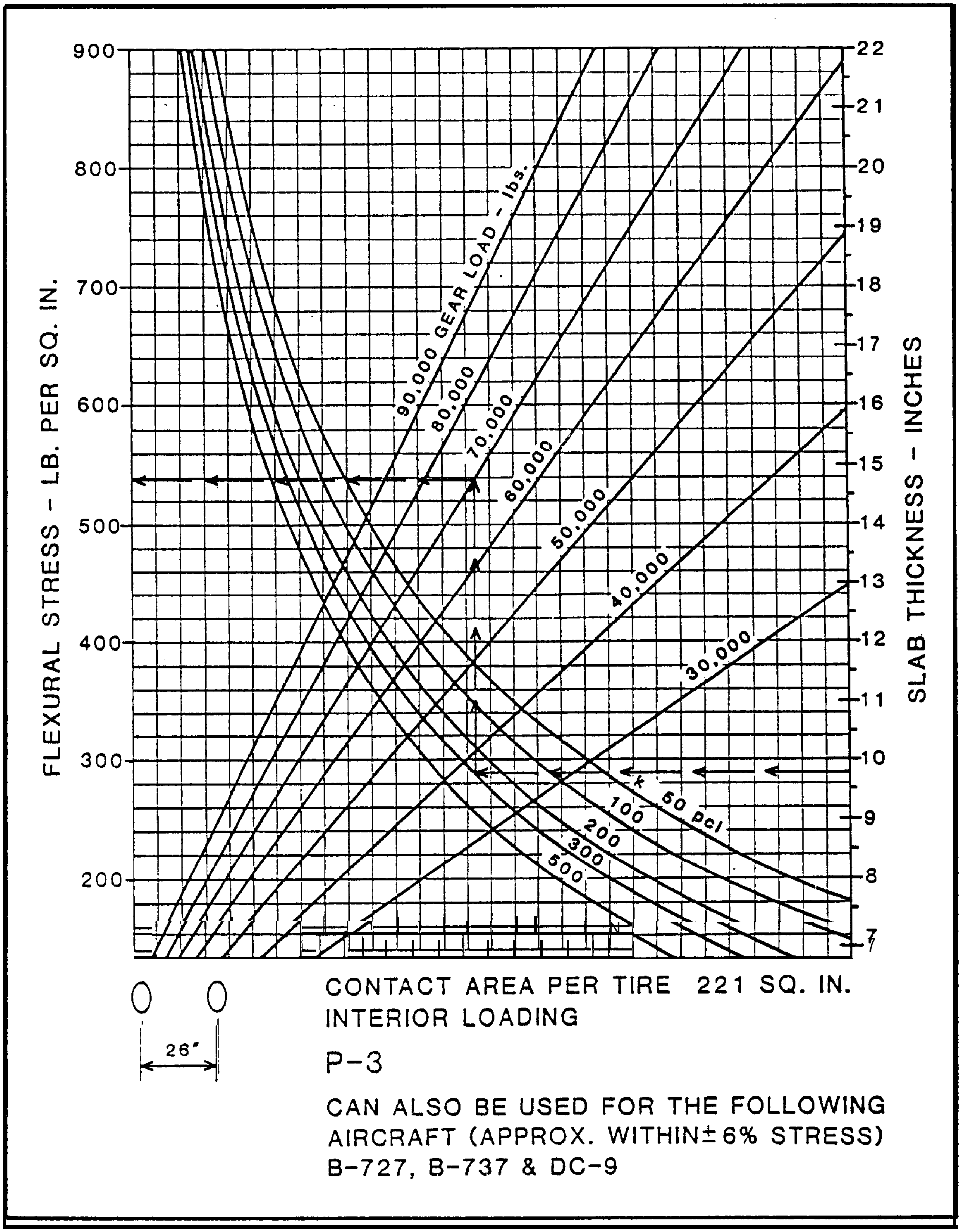


Figure 10
Chart for Determining Flexural Stress for P-3 Aircraft

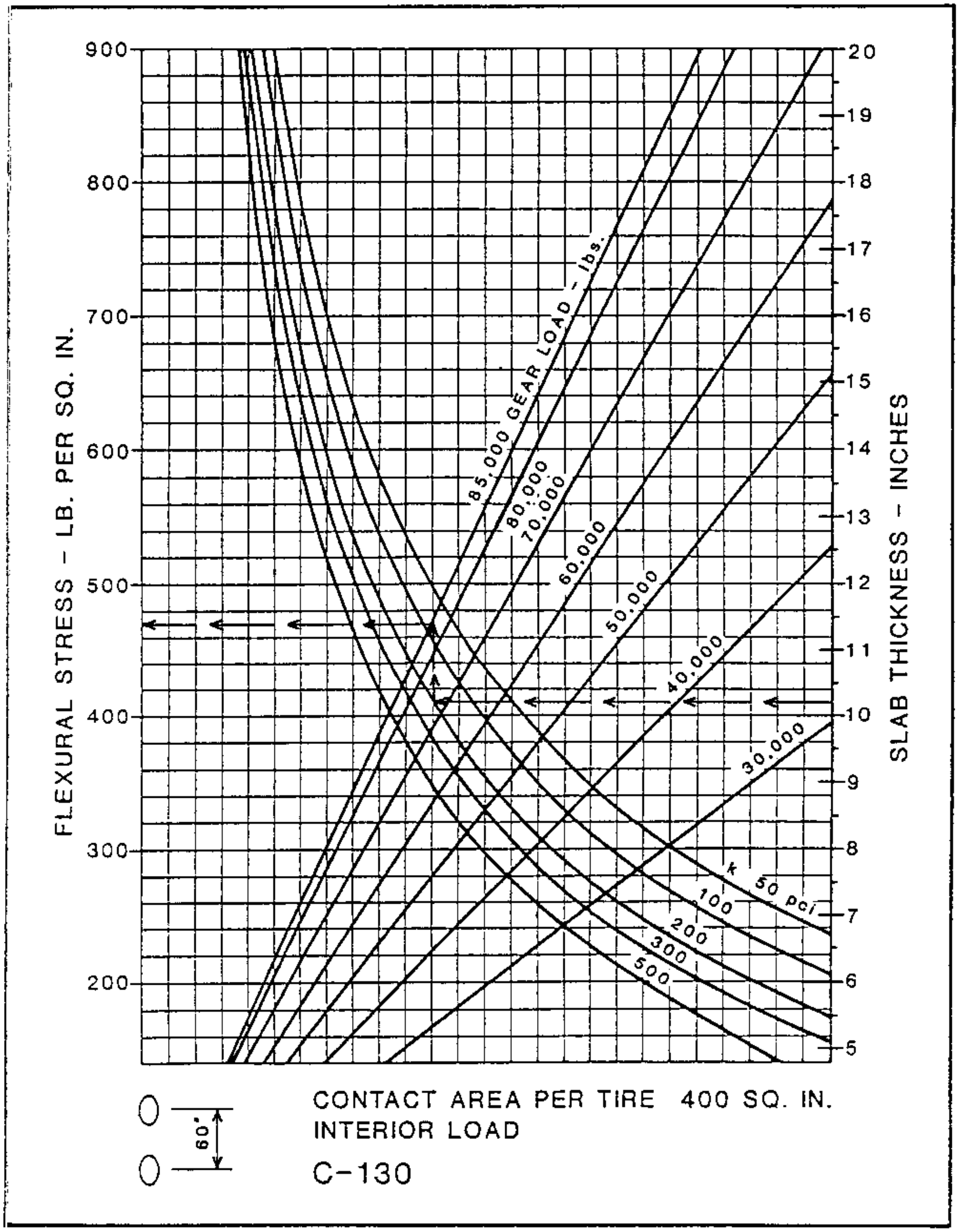
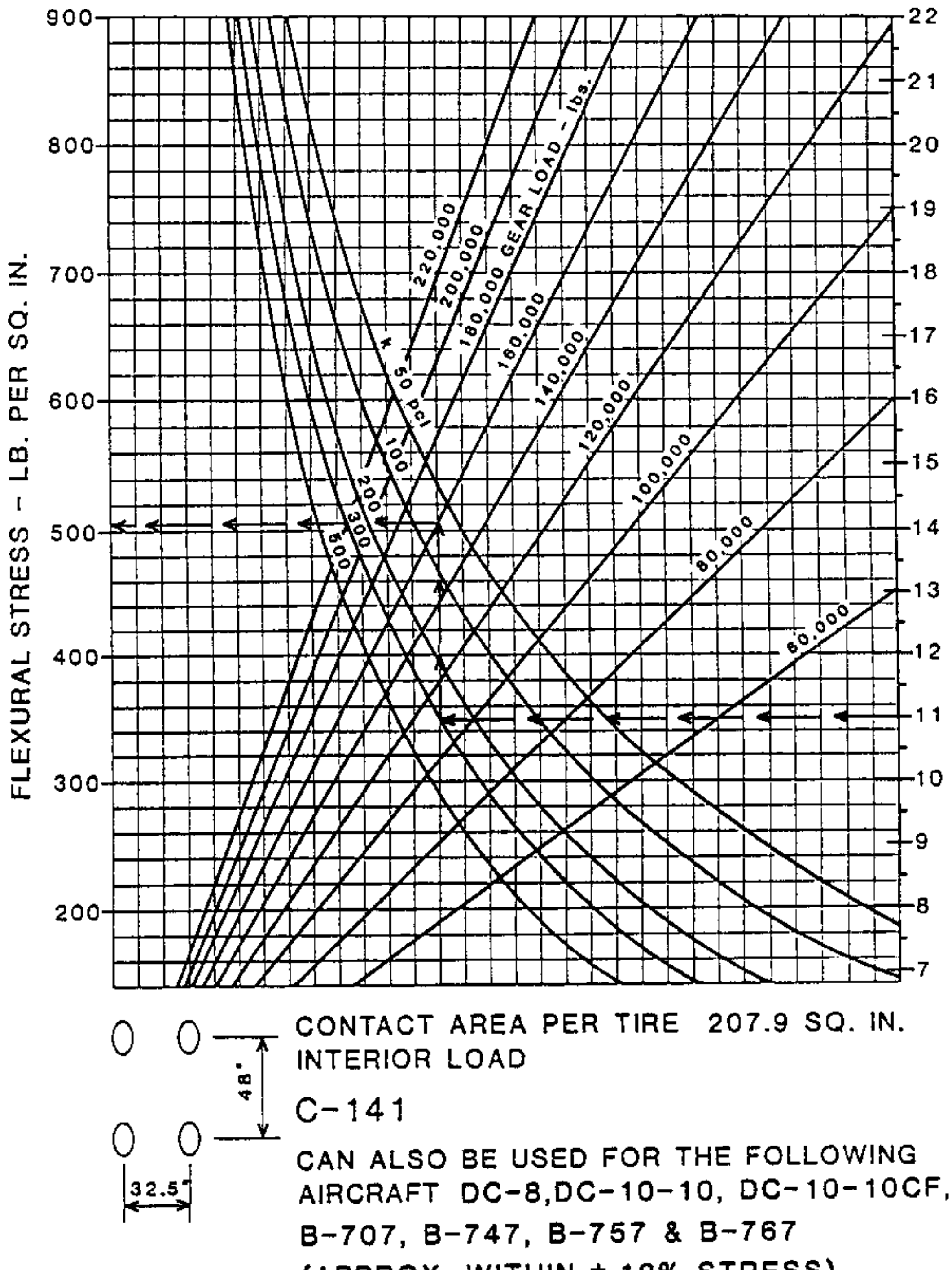


Figure 11
Chart for Determining Flexural Stress for C-130 Aircraft



CONTACT AREA PER TIRE 207.9 SQ. IN.
INTERIOR LOAD

C-141

CAN ALSO BE USED FOR THE FOLLOWING AIRCRAFT DC-8,DC-10-10, DC-10-10CF, B-707, B-747, B-757 & B-767 (APPROX. WITHIN ± 12% STRESS).

Figure 12
Chart for Determining Flexural Stress for C-141 Aircraft

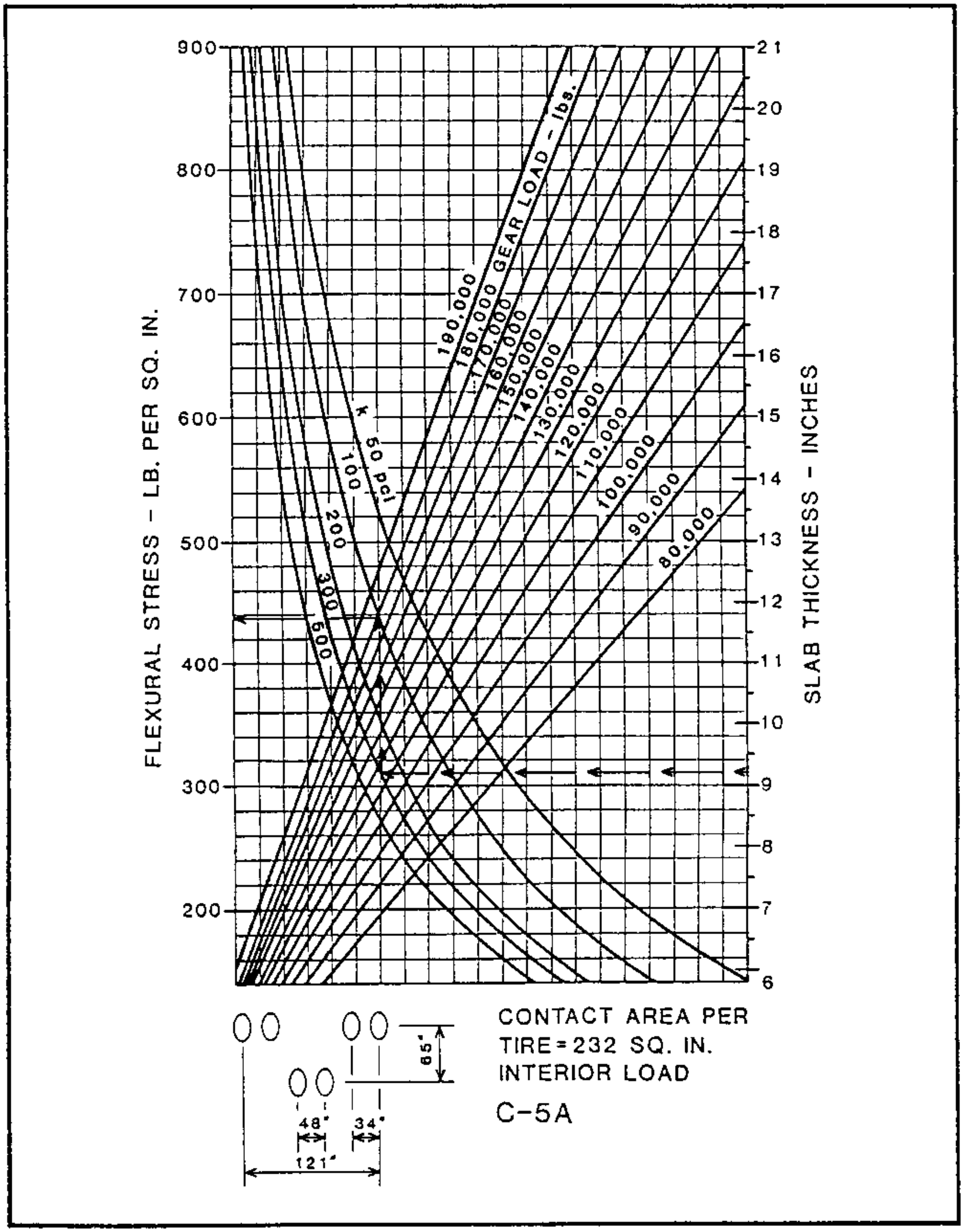


Figure 13
Chart for Determining Flexural Stress for C-5A Aircraft

8.6 Detailed Design Example. A detailed design example follows.

8.6.1 Background Information. A new runway is to be designed to serve frequent operations of C-141, C-130, and C-5A aircraft. In addition to these aircraft, the new facility will be used by F-14 and P-3 aircraft. The runway is located in a warm climatic region where frost penetration does not need to be considered in the design processs. The new runway is assumed to be 200 feet (60 m) wide and 10,000 feet (3050 m) in length. Use the following general design procedure when designing the rigid pavement for this runway.

8.6.2 Subgrade Evaluation and Testing. A subgrade investigation was performed to evaluate the support of the subgrade soil. Prior to the actual field survey, previous soils investigations, soils maps, climatic data, etc., were collected to provide background information on the soil conditions. Soil borings were then obtained to aid in evaluating the physical properties of the soil. Soil borings were taken at 200-foot intervals along the location of the proposed runway consistent with the recommendations given in Table 1.

Soil tests show that the subgrade soil can be classified as CL according to the Unified Soil Classification System. Test results show that there is no significant soil variation in the area for the new runway. Swelling soils are not a problem at the site.

Plate load bearing tests were performed according to ASTM D1196 to determine the modulus of subgrade reaction (k value). Because of the uniform soils throughout the area, only three plate load tests were taken. The results are summarized below:

Test Number	k Value, pci	(kg/mÀ3Ù)
1	100	(2 768 000)
2	150	(4 152 000)
3	130	(3 598 000)

Average = 127 pci (3 506 000 kg/mÀ3Ù)

Because of the uniform soil conditions throughout the site, a design k value of the average of the three tests, or 127 pci (3 506 000 kg/mÀ3Ù), is used.

8.6.3 Base Course Design. Results of a field survey and soil tests indicate that the subgrade soil has a high degree of saturation and low permeability. Thus, very little bottom drainage is likely. Therefore, a base material that is resistant to the detrimental effects of moisture should be used. A free-draining granular base course may be used to increase the subgrade k value to the minimum acceptable k value of 200 pounds per cubic inch (5 536 000 kg/m^3) on top of the base course. According to Figure 2 in Section 5, an 11 inch (279 mm) granular base course will raise the k value on top of the base to 200 pci (5 536 000 kg/m^3). To prevent intrusion of subgrade fines into the base course, a filter course is included in the design.

8.6.4 Traffic Projections. The following tables summarize the projected traffic for the new runway over a 20-year design period and the design gear loads. The pass-coverage ratios are obtained from Table 15 in Section 7.

Aircraft	Passes 20 years	Pass-Coverage Ratio Channelized	Unchannelized	Coverages, 20 years Channelized	Unchannelized
c-141	25,000	3.49	6.25	7,163	4,000
c-130	50,000	4.36	8.56	11,468	5,841
C-5A	25,000	1.62	2.20	15,432	11,364
P-3	50,000	3.45	6.49	14,493	7,704
F-14	100,000	8.58	17.00	11,655	5,882

Aircraft	Design Gear Load pounds	(kg)
c-141	155,000	(70 000)
c-130	84,000	(38 000)
C-5A	190,000	(86 000)
P-3	68,000	(31 000)
F-14	30,000	(13 600)

The design tire pressure for the F-14 is 240 psi (3 447 000 Pa).

8.6.5 Slab Thickness and Joints. The k value of 200 pounds per cubic inch (5 536 000 kg/m^3) as determined above and a design flexural strength of 650 pounds per square inch (4 482 000 Pa) are used to determine the required slab thickness. Results of the mixed traffic analysis for the channelized traffic areas are summarized in Table 20. Results for the unchannelized traffic areas are summarized in Table 21, This shows that a 13.8 inch (351 mm) concrete slab is required in areas of channelized traffic to serve the projected aircraft over a design life of 20 years. This is rounded up to a recommended thickness of 14.0 inches (356 mm). A 13.0 inch (330 mm) concrete slab is required in areas with unchannelized trafffic.

TABLE 20
Design Example for Primary (Channelized) Traffic Areas

PAVEMENT IDENTIFICATION **NEW E-W RUNWAY** TRAFFIC AREA **CHANNELIZED**
SLAB THICKNESS: **13.8** SINGLE WHEEL AIRCRAFT TIRE PRESSURE, psi
BASE K: **200 PCI** 1. **F-14** 1. **240**
FLEXURAL STRENGTH **650 PSI** 2. 2.

(1) AIRCRAFT	(2) DESIGN GEAR LOAD	(3) PROJECTED PASSES	(4) P/C	(5) PROJECTED COVERAGES (n)	(6) INTERIOR STRESS	(7) STRESS/F S	(8) ALLOWABLE COVERAGES (N)	(9) FATIGUE LIFE CONSUMED (n/N)
C-141	155,000	25,000	3.49	7,163	400	0.62	18,000	0.40
C-130	84,000	50,000	4.36	11.468	300	0.46	1,700,000	0.01
C-5	190,000	25,000	1.62	15,432	340	0.52	300,000	0.05
P-3	68,000	50,000	3.45	14,493	330	0.51	400,000	0.04
F-14	30,000	100,000	8.85	11,655	230	0.35	UNLIMITED	-
							$\sum n/N=$	0.50

TABLE 21
Design Example for Secondary (Unchannelized) Traffic Areas

PAVEMENT IDENTIFICATION **NEW E-W RUNWAY** TRAFFIC AREA **UNCHANNELIZED**
SLAB THICKNESS: **12.5"** SINGLE WHEEL AIRCRAFT TIRE PRESSURE, psi
BASE K: **200 PCI** 1. **F-14** 1. **240**
FLEXURAL STRENGTH **650 PSI** 2. 2.

(1) AIRCRAFT	(2) DESIGN GEAR LOAD	(3) PROJECTED PASSES	(4) P/C	(5) PROJECTED COVERAGES (n)	(6) INTERIOR STRESS	(7) STRESS/F S	(8) ALLOWABLE COVERAGES (N)	(9) FATIGUE LIFE CONSUMED (n/N)
C-141	155,000	25,000	6.25	4,000	455	0.70	2,000	2.00
C-130	84,000	50,000	8.56	5,841	350	0.54	180,000	0.03
C-5	190,000	25,000	2.20	11,364	370	0.57	75,000	0.15
P-3	68,000	50,000	6.49	7,704	390	0.60	32,000	0.24
F-14	30,000	100,000	17.0	5,882	270	0.42	UNLIMITED	-
							$\sum n/N=$	2.42

Table 21 (Continued)
Design Example for Secondary (Unchannelized) Traffic Areas

PAVEMENT IDENTIFICATION **NEW E-W RUNWAY** TRAFFIC AREA **UNCHANNELIZED**
SLAB THICKNESS: **13.0"** SINGLE WHEEL AIRCRAFT TIRE PRESSURE, psi
BASE K: **200 PCI** 1. **F-14** 1. **240**
FLEXURAL STRENGTH **650 PSI** 2. 2.

1 AIRCRAFT	2 DESIGN GEAR LOAD	3 PROJECTED PASSES	4 P/C	5 PROJECTED COVERAGES (n)	6 INTERIOR STRESS	7 STRESS/F S	8 ALLOWABLE COVERAGES (N)	9 FATIGUE LIFE CONSUMED (n/N)
C-141	155,000	25,000	6.25	4,000	430	0.66	6,000	0.67
C-130	84,000	50,000	8.56	5,841	330	0.51	400,000	0.01
C-5	190,000	25,000	2.20	11,364	360	0.55	130,000	0.09
P-3	68,000	50,000	6.49	7,704	370	0.57	75,000	0.10
F-14	30,000	100,000	17.0	5,882	260	0.40	UNLIMITED	-
							Σ n/N:	0.87

Section 9: JOINT DESIGN FOR NON-REINFORCED CONCRETE PAVEMENT

9.1 Introduction. The following sections describe the joint designs to be used for non-reinforced concrete pavements. See NFGS-02522, Joints, Reinforcement and Mooring Eyes in Concrete Pavements, for guide specifications on joints and reinforcement.

9.2 Basis for Design. Use joints to:

a) limit curling and warping stresses in the pavement which are due to temperature and moisture gradients through the slab,
b) prevent and control cracking due to volume changes in the concrete,
c) prevent damage to immovable structures,
d) facilitate construction.

9.3 Types of Joints and Uses.

9.3.1 Expansion Joints. Expansion joints allow for the expansion of the pavement and the reduction of high compressive stresses at critical locations in the concrete pavement in hot weather. Expansion joints are placed the full depth of the slab. Use expansion joints at all intersections of pavements with fixed structures, at nonperpendicular pavement intersections and between existing and new concrete pavements when the joints in the adjacent slabs are not aligned. Expansion joints are not otherwise required within the non-reinforced concrete pavement. See Figure 14 for expansion joint details.

9.3.2 Contraction (Weakened Plane) Joints. Use contraction joints to: (1) control cracking in the pavement due to volume changes resulting from a temperature decrease or a moisture decrease, and (2) limit curling and warping stresses from temperature and moisture gradients in the pavement. Form contraction joints in concrete by partial depth sawing or by installing sawable inserts. The saw cut joint or formed groove provides a weakened plane which will crack through the full slab depth during shrinkage and contraction of the concrete as it cures. Contraction joints are required in the transverse direction and also in the longitudinal direction depending upon slab thickness and spacing of the construction joints. See Figure 15 for contraction joint details.

9.3.3 Construction Joints. Construction joints are used between paving lanes or when abutting slabs are placed at different times. Longitudinal and transverse construction joints may be required. Transverse construction joints will be required when it is necessary to stop concrete placement for a length of time sufficient to allow the concrete to begin to set. Longitudinal construction joints are generally spaced 20 to 50 feet (6 to 15 m) apart depending on the construction equipment.

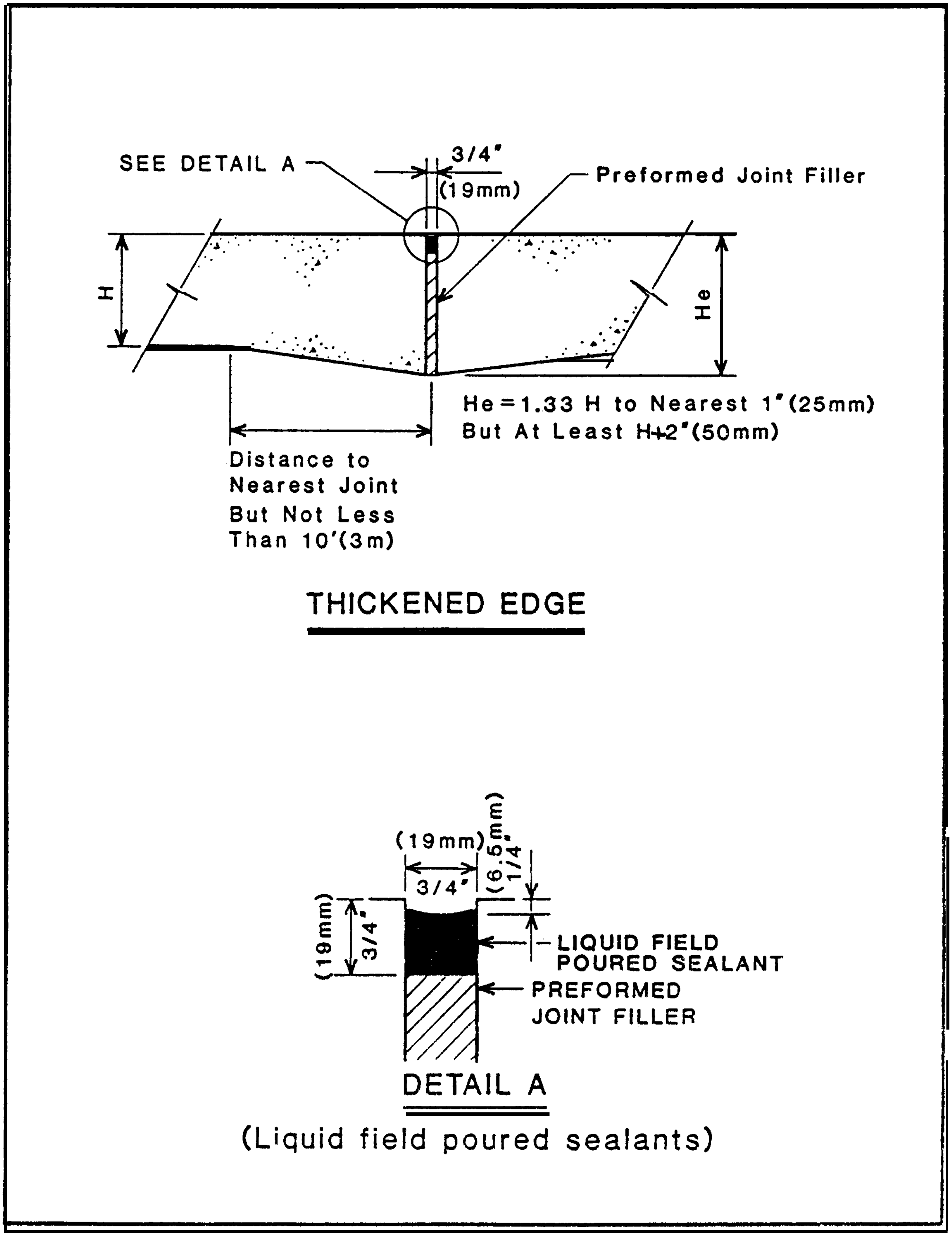
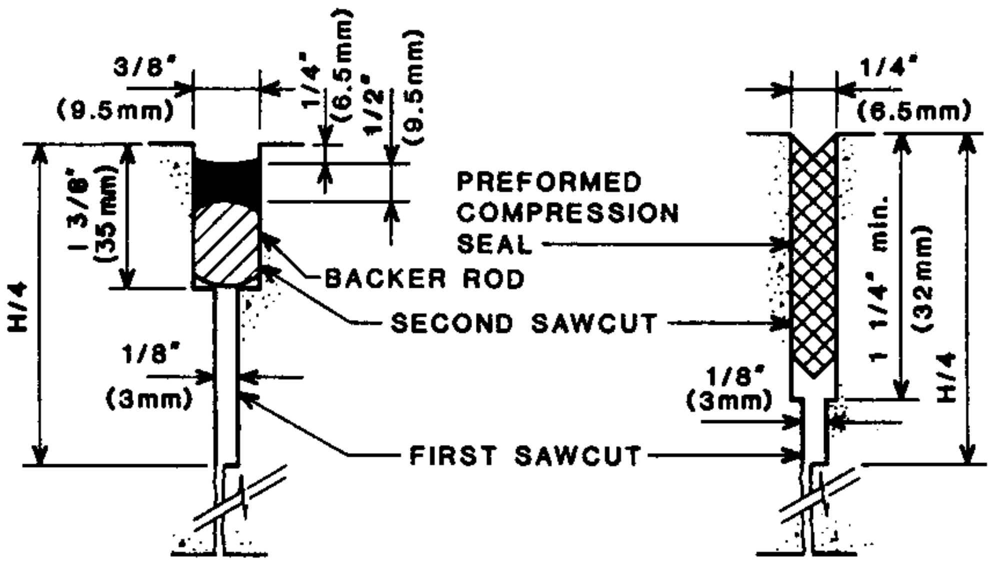
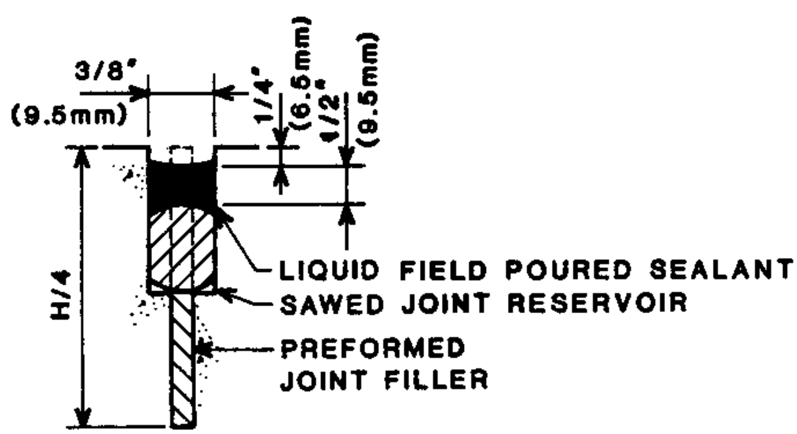


Figure 14
Expansion Joint Details



NOTE: FOR BONDED CONCRETE OVERLAY, JOINTS MUST BE SAWED COMPLETELY THROUGH OVERLAY.

DETAILS OF SAWED GROOVES



SAWABLE INSERT

DETAIL OF FORMED GROOVE

Figure 15
Contraction (Weakened Plane) Joint Details

9.3.3.1 Transverse Construction Joints. Locate all transverse construction joints at the same location as regularly spaced transverse joints. Provide for load transfer or a thickened edge.

9.3.3.2 Longitudinal Construction Joints. Construct longitudinal construction joints as shown in Figure 16 and indicated below.

a) Keyed Joint. Keyways have been used extensively to provide load transfer along longitudinal joints. However, there has been a substantial amount of keyway failure under heavy aircraft loading on thinner slabs. Keyed joints may only be used on slabs 9 inches (229 mm) thick or greater.

b) Butt Joint. A butt joint may be used for longitudinal construction joints on pavements less than 9 inches (229 mm) thick constructed with a stabilized base.

c) Thickened Edge Joint. A thickened edge joint may be used for longitudinal construction joints. The thickened edge joint may be used for any pavement thickness and base type.

9.4 Joint Spacing. The standard slab size for pavements is 12.5 by 15 feet (3.8 by 4.6 m). Transverse joint spacing is 15.0 feet (4.6 m) and longitudinal joint spacing is 12.5 feet (3.8 m). For slabs having a thickness greater than 12 inches (305 mm), joint spacing can be increased to a maximum of 20 feet (6.1 m). The transverse joint spacing shall not vary from the longitudinal joint spacing by more than 25 percent. Figure 17 shows standard joint spacings.

9.5 Load Transfer Design. A properly designed joint must provide adequate load transfer across the joint. Load transfer efficiency is normally defined as the ratio of deflection of the unloaded side to the deflection of the loaded side of the joint. Good load transfer will aid in preventing deterioration such as corner breaks, transverse and longitudinal cracking, faulting, pumping, and spalling. Different amounts of load transfer can be obtained through the use of aggregate interlock, dowel bars, keyways, a stabilized base or a combination of these.

9.5.1 Aggregate Interlock. Aggregate interlock can provide adequate load transfer across joints when the pavement is originally constructed, or during hot weather. However, as joint movements due to temperature variation and load applications increase, and the joint begins to open, aggregate interlock is lost and load transfer is greatly reduced. The effectiveness of aggregate interlock may be improved by increasing base strength and the angularity of coarse aggregate.

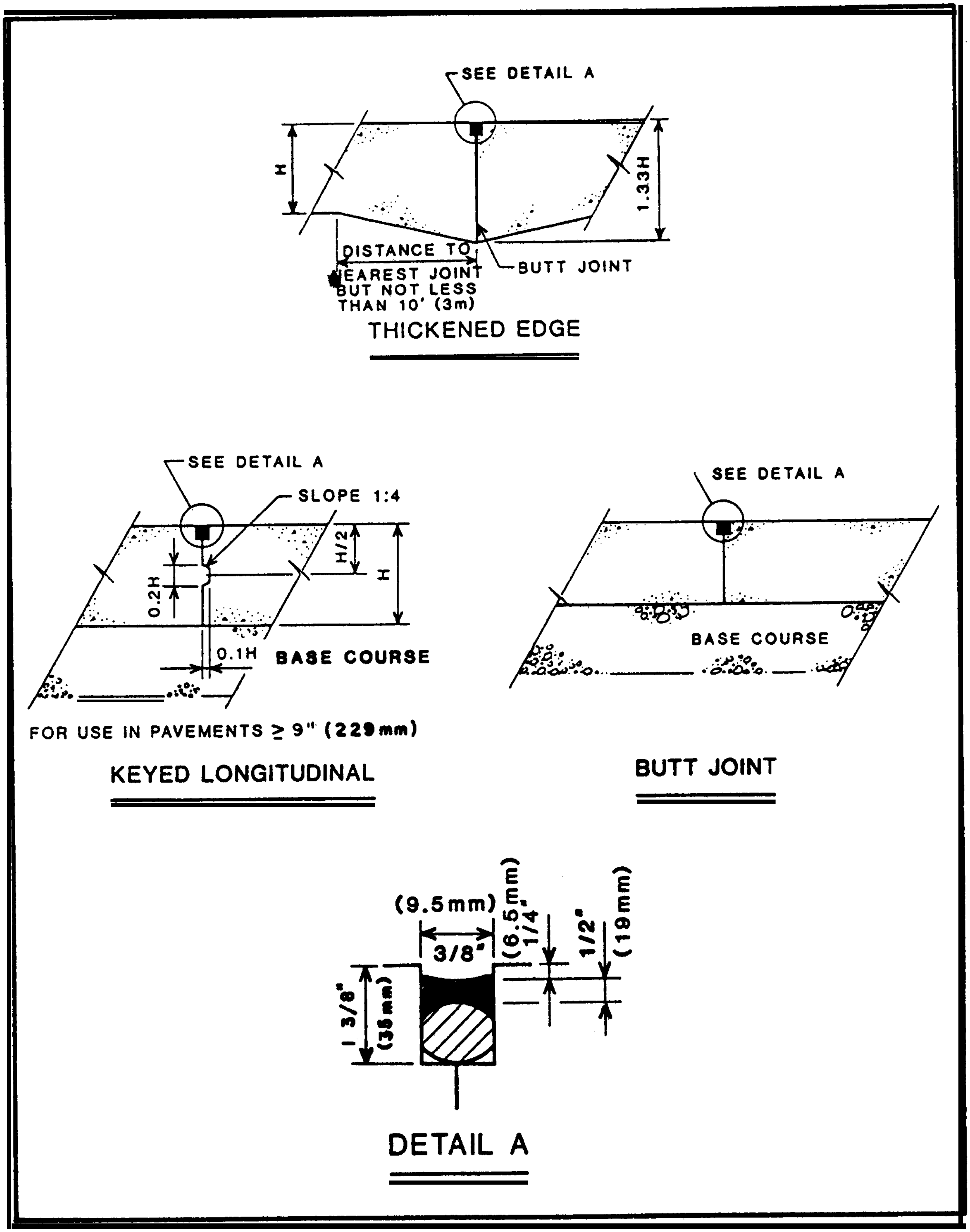


Figure 16
Construction Joint Details

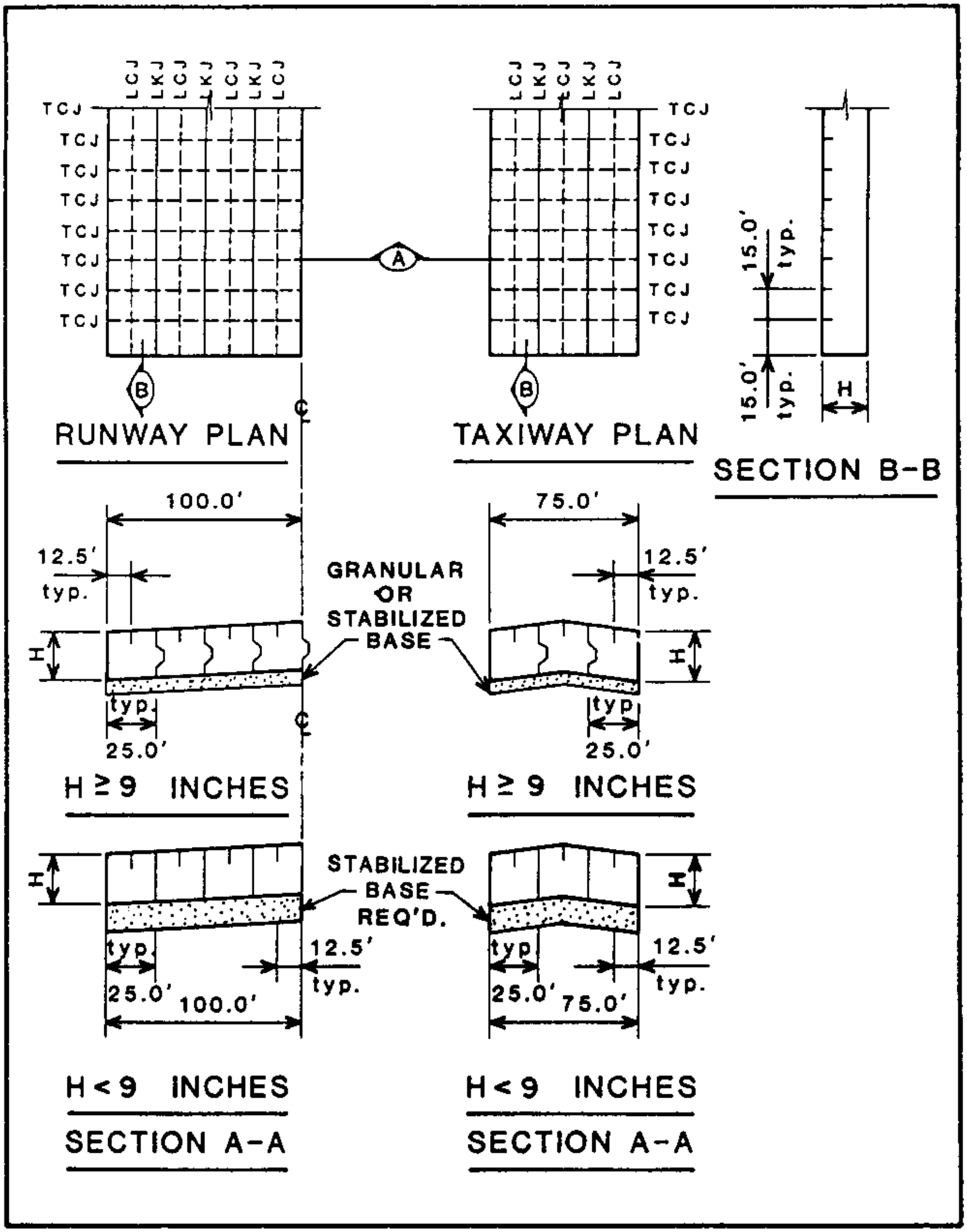


Figure 17
Typical Longitudinal and Transverse Joint Layout

9.5.2 Dowel Bars. Dowel bars are used to provide load transfer and prevent excessive vertical displacements of adjacent slabs. Dowels are to be avoided for new construction because of a history of quality control problems (such as poor alignment). There are some situations where the use of dowels is appropriate, such as for creating load transfer where tying in to existing pavements.

9.5.3 Keyways. Keyways may be used to provide load transfer along longitudinal construction joints. Do not use keyways in pavements less than 9 inches (229 mm) thick because the small keyways have limited strength and may crack under heavy aircraft loadings. Small keyways may also be difficult to construct because coarse aggregate may not be able to enter the key and segregation of the material may occur. Keyways may be used for pavements 9 inches (229 mm) thick and greater. However, keyway failures have caused serious problems.

9.5.4 Stabilized Base. A stabilized base can be used to improve load transfer effectiveness by reducing joint deflections through increased support across a joint. Use a stabilized base for all pavements less than 9 inches (229 mm) thick to provide improved load transfer and lower deflections and stresses. A stabilized base may also be used for pavements greater than 9 inches (229 mm) thick to provide additional load transfer. Where thickened edge joints are used, the stabilized base is not required.

9.6 Joint Sealants. Joint sealants are used to provide a seal to reduce infiltration of water and incompressibles. An effective joint seal will help retard and reduce distress related to free water and incompressibles, such as pumping, spalling, faulting, and corrosion of mechanical load transfer devices. See NFGS-02522 for specifications on joint sealing compounds.

Several pavement areas require fuel-resistant or blast-resistant joint sealants. Use jet fuel resistant sealants for all aprons. Use blast resistant sealants for the first 1000 feet (305 m) of runways and exits at runway ends. Use sealing compounds meeting Federal Specification SS-S-1401C, Sealant, Joint, Non-Jet-Fuel-Resistant, Hot-Applied, For Portland Cement and Asphalt Pavements, for taxiways and runway interiors. Specific pavement areas are detailed in NFGS-02522.

9.6.1 Types of Sealant Materials. The three major types of sealant materials are (1) field poured, hot applied; (2) field poured, cold applied; and (3) preformed compression seals. These materials may be jet fuel resistant (tar based) or non-jet fuel resistant (typically asphalt based).

9.6.1.1 Field Poured, Hot Applied. This group of sealants includes rubberized asphalt sealant and rubberized tar sealant. Rubberized asphalt joint sealants must meet Federal Specification SS-S-1401C and may be used in the areas designated in NFGS-02522. Rubberized tar sealants must meet Federal Specification SS-S-1614A, Sealant, Joint, Jet-Fuel-Resistant, Hot-Applied, For Portland Cement and Tar Concrete Pavements.

9.6.1.2 Field Poured, Cold Applied. These are two-component, polymer-type, cold-applied heat and jet fuel-resistant joint sealants. These sealants must meet Federal Specification SS-S-200E, Sealant, Joint, Two-Component, Jet-Blast-Resistant, Cold-Applied, For Portland Cement Concrete Pavements.

9.6.1.3 Preformed Compression Seals. The most common type of preformed compression seal is the neoprene compression seal. Neoprene compression seals must satisfy ASTM D2628. Preformed compression seals may be used in the areas designated in NFGS-02522. Preformed compression seals are designed to be in compression for their entire life. There is no bond between the compression seal and the sidewalls of the joint to sustain tension.

9.6.2 Joint Reservoir Design. The joint reservoir must be properly designed so that the joint sealant can withstand compressive and tensile strains.

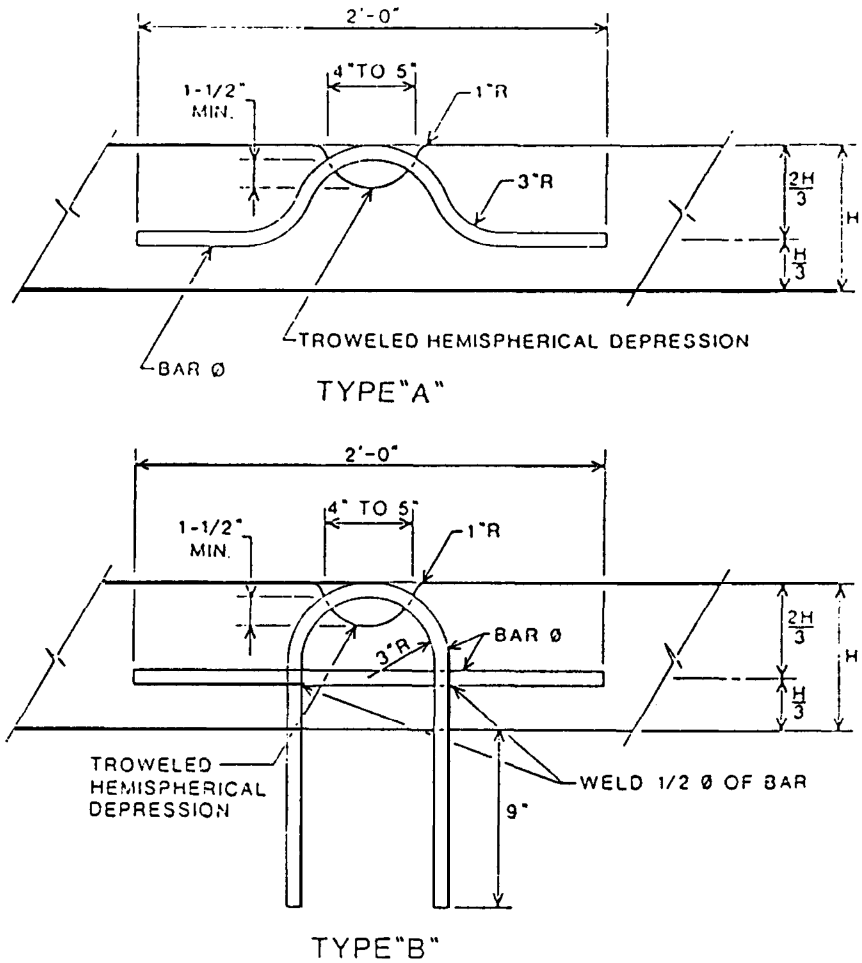
9.6.2.1 Field Poured Sealants. The shape factor, which is defined as the ratio of the depth of the sealant to the width of the joint, should be between 1.0 and 1.5. Dimensions of the joint sealant and reservoir are shown in Figure 15. A backer rod or bond breaking tape must be used to help obtain a proper shape factor and to prevent the joint sealant from bonding to the bottom of the joint reservoir. Most field-poured liquid joint sealants can withstand strains of approximately 25 percent of their original width. Joint reservoir and sealant dimensions shown in Figure 15 are based on a slab size of 12.5 feet (3.8 m) by 15.0 feet (4.6 m).

9.6.2.2 Preformed Compression Seals. The reservoir width for preformed compression seals must be designed to keep the sealant in compression at all times. The depth of the reservoir must exceed the depth of the seal, but is not related directly to the width of the joint. The width of the compression seal should be approximately twice the width of the joint. The limits on the compression seal are normally 20 percent minimum and 50 percent maximum compression strain of the original sealant width. For example, the working range of a 1-inch (25 mm) wide neoprene compression seal is from 0.5 to 0.8 inches (12 to 20 mm).

If the seal is subjected to compression greater than the 50 percent level for extended periods of time, the seal may take a compression set, and the webs may bond to each other. If this happens, the seal will not open as the joint opens, and the seal will no longer be effective.

The joint dimensions for the standard size slab are shown in Figure 15. Design sealant dimensions based on the actual joint spacing. Choose preformed neoprene compression seal dimensions so that the working range of the joint is within the working range of the sealant.

9.7 Tiedown Mooring Eyes. Tiedown mooring eyes are required in the center of each slab over the entire aircraft parking apron except on peripheral taxi lanes. See Figure 18 for mooring eye details.



BAR SIZES FOR TYPES A&B	
H	BAR Ø
<10"	3/4"
10" TO 12"	1"
13" TO 16"	1-1/4"

NOTES:

1. PLACE MOORING EYES IN THE CENTER OF EACH 12.5' BY 15.0' SLAB OVER ENTIRE SURFACE OF PARKING AREA PAVEMENTS UNLESS OTHERWISE INDICATED.
2. PLACE MOORING EYES IN HANGAR FLOORS AS DETERMINED BY PROJECT REQUIREMENTS.

Figure 18
Tiedown Mooring Eye Details

THIS PAGE IS INTENTIONALLY BLANK

Section 10: DESIGN OF REINFORCED PAVEMENTS

10.1 Introduction. The structural design of non-reinforced concrete pavements is presented in Sections 8 and 9. Airfield pavements can also be designed using reinforced, continuously reinforced and prestressed concrete pavements. The following sections briefly describe the basis of design for each of these types of pavements. Existing design procedures are referenced.

These designs will not normally be used for Navy and Marine Corps projects but may be considered for special or unusual design conditions on a case-by-case basis and must be approved by the Naval Facilities Engineering Command. The exception to this is in odd-shaped slabs and mis-matched joints where reinforcing is required. The selection of the final pavement design should be based upon a life-cycle cost analysis and other critical factors.

10.2 Basis of Design.

10.2.1 Jointed Reinforced Concrete Pavement. This pavement type utilizes longer joint spacing than non-reinforced jointed pavement. The cracks that develop from shrinkage, warping, curling, and traffic load stresses are held together by reinforcement. Steel reinforcing is used to slow the deterioration of cracks that develop in the concrete slab by holding these cracks tightly together to maintain aggregate interlock. See Air Force Manual 88-6, Rigid Pavements for Airfields Other Than Army, for additional information and design procedures for jointed reinforced concrete pavement.

10.2.1.1 Thickness. The thickness design for jointed reinforced concrete pavement is similar to jointed plain concrete pavement design, modified by the results of accelerated traffic tests. These tests demonstrate that the required pavement thickness may be less than the required thickness of a non-reinforced jointed concrete pavement that provides equal performance. However, as thickness is reduced substantially, premature distress may occur. Therefore, because of inconsistent performance of thin reinforced pavements, for new construction, the thickness shall not be reduced from that determined by the design procedure given in Section 8.

10.2.1.2 Reinforcement. Reinforcing steel is usually required in both the transverse and longitudinal directions. The steel may be deformed bars or welded wire fabric. Typical amounts of reinforcing range from 0.05 to 0.25 percent area.

10.2.1.3 Joints. The maximum slab size for jointed reinforced concrete pavements is a function of the slab thickness, yield strength of the reinforcing steel, and the percent of reinforcement. Slab size is commonly 25 feet (8 m) square.

All joints in reinforced concrete pavements, with the exception of keyways and thickened-edge joints, are doweled. Dowels are effective in providing load transfer. Alignment of the dowel bars and adequate consolidation around the dowel basket are critical factors.

10.2.2 Continuously Reinforced Concrete Pavement. No transverse joints are used in continuously reinforced pavements except at construction joints and pavement intersections. Steel is usually provided in both the longitudinal and transverse directions. Design of continuously reinforced concrete pavements is based on providing enough steel to hold all shrinkage cracks closely together and maintain aggregate interlock. The crack spacing ranges from as little as 2 feet (1 m) to as much as 12 feet (4 m) and is dependent upon the amount of reinforcing steel, friction between the slab and base course, and several other factors.

The primary advantages of using continuously reinforced pavements are that there are almost no joints, it is generally smoother, and it may be a low-maintenance pavement when properly designed and constructed. Failure in this pavement type is commonly considered to be closely spaced cracking which often leads to deterioration of cracks and spalling. These conditions eventually lead to punchouts. See Air Force Manual 88-6 for additional information and design procedures for continuously reinforced concrete pavement design.

10.2.2.1 Thickness. The design thickness of continuously reinforced concrete pavements is a function of the design flexural strength and modulus of elasticity of the concrete, modulus of subgrade reaction, and the allowable concrete flexural stress. As thickness is reduced, deflections and vertical stresses will increase which may result in premature failure. For new construction, the thickness shall not be reduced from that determined by the design procedure given in Section 8.

10.2.2.2 Reinforcement. The amount of reinforcement is chosen to provide optimum crack spacing and width. The required amount of longitudinal reinforcement is a function of the following: (a) yield strength of the reinforcing steel, (b) flexural strength of the concrete, (c) average temperature differential in the pavement, and (d) modulus of elasticity of the reinforcing steel in tension. The amount of reinforcing steel required in the transverse direction is a function of the following: (a) the width of the pavement slab, (b) the yield strength of the reinforcing steel, and (c) the friction factor between the pavement and the underlying base material. Typical percentages of reinforcement range from 0.5 to 0.75 percent area.

10.2.2.3 Joints. Longitudinal construction joints are required and are generally a function of the construction equipment used and the width of the paving lanes. Transverse joints are placed only at construction joints. Continuity at transverse construction joints is provided by continuing the longitudinal steel through the slab.

10.2.3 Steel Fiber Reinforced Concrete Pavement. This section is intended to provide interim guidance on the use of steel fiber reinforced concrete and will be revised at a later date. Steel fiber reinforced concrete pavements consist of Portland cement concrete containing discontinuous discrete steel fibers. Typical fiber contents are in the range of 1.0 to 2.0 percent by volume. The addition of the steel fibers to the concrete mix increases the flexural strength of the pavement. The higher flexural strengths permit steel fiber reinforced pavements to be designed and constructed somewhat thinner than non-reinforced concrete pavements. However, because of the added cost of the steel fiber, there is no clear economic advantage in using steel fiber reinforced concrete for new pavement.

Steel fiber reinforced concrete should not be used in the design of new airfield pavements. Steel fiber reinforced concrete may be considered for overlays where clearance requirements or grade changes dictate an overlay which is thinner than non-reinforced concrete will permit. See Section 11 for additional information.

10.2.4 Prestressed Concrete Pavement. Prestressed pavements utilize the fact that concrete is much stronger in compression than in tension. The prestressing operation creates a compressive stress in the concrete prior to the application of a live load, and will tend to decrease the difference between the tensile stresses resulting from the load and the compressive stresses. This allows the pavement thickness to be reduced and still provide similar performance to a thicker non-reinforced concrete pavement. The primary advantages to using a prestressed concrete pavement are: (1) reduction of the number of transverse joints, (2) elimination of cracks in the pavement surface, and (3) reduction in the required pavement thickness. See Air Force Manual 88-6 for additional information and design procedures for prestressed concrete pavement.

10.2.4.1 Thickness. The thickness of prestressed concrete pavements is primarily a function of the level of prestress and traffic loading. The required level of prestress is based on the maximum combined stress due to load, curling, and friction. The minimum thickness for prestressed pavements is 6 inches (150 mm).

10.2.4.2 Reinforcement. Prestressing tendons are required in both the transverse and longitudinal directions. The size and spacing of the tendons is a function of the required prestress level.

10.2.4.3 Joints. The maximum length of prestressed concrete slabs is 500 feet (150 m). The spacing between the longitudinal joints is controlled by the type of construction equipment and is generally a minimum of 25 feet (8 m. Because of the larger slab length, transverse joints must be adequately designed to accommodate the larger movements due to temperature change.

Section 11: STRENGTHENING OF RIGID PAVEMENTS

11.1 General. Strengthening of a pavement increases its load-bearing capacity, and is accomplished with an overlay of either asphalt concrete or Portland cement concrete. Airfield pavements require strengthening to support heavier and/or more numerous aircraft loadings than those for which they were originally designed. Strengthening is also performed on pavements which have been damaged by a combination of traffic loadings and climatic influences to the extent that they can no longer support the traffic expected to use them. This section provides guidance and criteria for the design of flexible and rigid overlays of rigid airfield pavements to increase structural capacity. Functional overlays, those which improve operational condition of the airfield pavement but do not significantly increase its structural capacity, are also discussed.

11.2 Rehabilitation Alternative Selection. A structural improvement such as an overlay should be performed only after it has been determined that the pavement possesses a structural deficiency for which an overlay is an appropriate and cost-effective method of rehabilitation. The major steps in the overall pavement rehabilitation alternative selection process are:

a) Office data collection: for pavement age, design, materials and soils properties at the time of construction, traffic data, climate records, and maintenance history.

b) Field and lab data collection and testing: for present structural and functional condition, surface and subsurface drainage conditions, vertical grade, and in situ materials and soils properties.

c) Definition of the pavement deterioration problem: in terms of the types of deterioration present and the mechanisms responsible for the deterioration.

d) Development of feasible rehabilitation alternatives: which both repair the existing distress and prevent its recurrence, as much as possible, by effectively addressing the causes of the deterioration.

e) Selection of the preferred alternative: which is the most appropriate and most cost-effective method of rehabilitation for the pavement, given existing constraints (e.g., funding, allowable closure time).

11.2.1 Pavement Evaluation Procedure. A comprehensive evaluation of the present condition of the pavement must be performed to determine an accurate and complete definition of the pavement deterioration problem.

11.2.1.1 Condition Survey. Use the Pavement Condition Index (PCI) rating procedure for performing the visual distress survey and condition rating of Navy and Marine Corps airfield pavements. The PCI procedure has been adopted by the U. S. Navy for airfield pavement condition rating.

a) Description of PCI. The PCI is a number from 0 to 100 which reflects the structural and functional condition of a pavement as it would be rated subjectively by a panel of experienced airfield pavement engineers. Its scale and associated condition ratings are shown in Figure 19. The PCI is calculated from data collected during a visual distress survey in which pavement distresses are quantified by type, amount, and severity. The mean PCI can be computed for any individual pavement. Procedures have been developed for performing the distress survey either by sampling a portion of the pavement surface or by inspecting the entire pavement area.

b) Overall Condition. The mean PCI of a pavement section describes its overall condition (e.g., fair, good, etc.) and thus is an indicator of the level of repair or rehabilitation work needed. The following are general guidelines for the level of repair or rehabilitation work that may be expected to be most cost-effective for a pavement with a given PCI:

Current PCI	Most Cost-Effective Rehabilitation (Within Next Two Years)
100 to 70	Preventive maintenance and restoration, including joint/crack sealing, undersealing (filling voids), slab replacement, full-depth repair, partial-depth spall repair.
69 to 40	Most cost-effective rehabilitation may range from preventive maintenance to major rehabilitation. Decision requires an engineering and a life-cycle cost analysis.
39 to 25	Major rehabilitation including overlays with or without keel replacement.
24 to 0	Major reconstruction (overlay is possible with extensive pre-overlay repair).

Note: There are exceptions to these guidelines, such as when a change in mission aircraft necessitates an increase in structural capacity regardless of the PCI value.

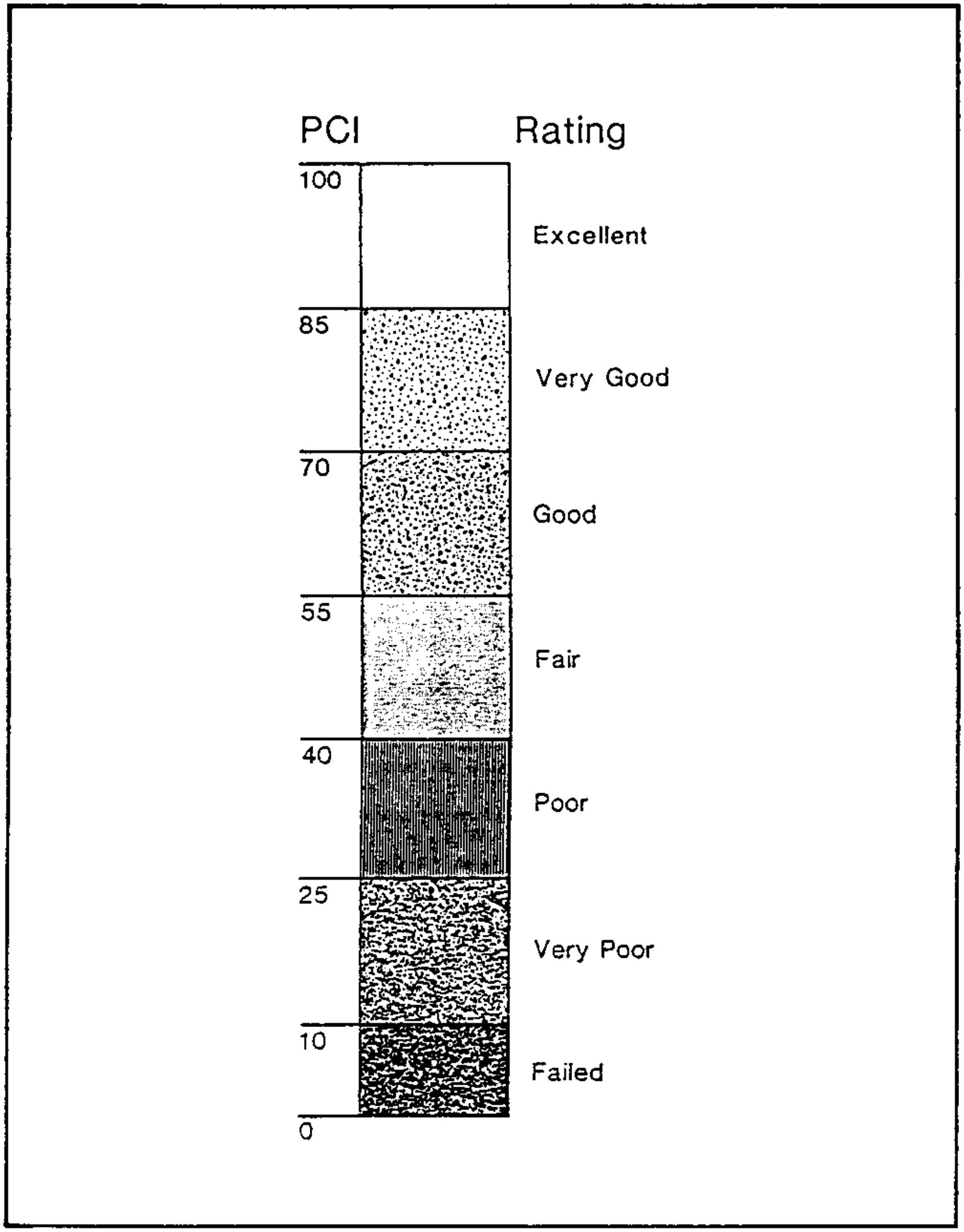


Figure 19
PCI Scale and Condition Rating

c) Variation of PCI Within a Pavement. Variation of the PCI (or cracking) within a pavement can be identified by plotting the PCI along the pavement's length (Figure 20) and across its width (Figure 21) at various locations along its length. Isolated locations of low PCI will be evident on the profile. These locations should receive localized repair (e.g., spa11 repair, slab replacement) to bring them up to the condition level of the rest of the pavement prior to rehabilitation of the entire pavement. Systematic variation of the PCI may be evident along the length or across the width of the pavement.

The example profiles in Figures 20 and 21 show that the ends of this runway are in poorer condition than the interior and the center keel section of the runway is in much poorer condition than the edges. Variation along the length of an airfield pavement should be corrected with localized repair, since an overlay of varying thickness is usually not feasible over a relatively short length of pavement such as a runway. Variation across the width of a pavement may be corrected either with localized repair or by varying the rehabilitation design.

d) Rate of Deterioration. Records of past PCI surveys, if available, can be used to determine the rate of deterioration of the pavement by plotting the PCI against its age in years, as illustrated in Figure 22. Take the rate of deterioration of the pavement into consideration in the life-cycle cost comparison of rehabilitation alternatives.

11.2.1.2 Distress Evaluation. Classify the types of distress observable in pavements as being related to traffic loads, climate and durability (including materials durability, temperature, and moisture), or other causes (e.g., swelling soils). Pavements exhibiting predominantly load-related distresses are, likely to have a structural deficiency and are candidates for structural rehabilitation by overlaying as described in this section.

Note whether or not the distresses present in the pavement appear to be aggravated by poor drainage conditions. If moisture is causing accelerated pavement deterioration, take steps to correct this before an overlay is constructed or other rehabilitation is performed. Make surface and subsurface drainage adequate, and properly seal joints and cracks.

11.2.1.4 Load-Bearing Capacity. Determine the pavement's load-bearing capacity. Procedures for determining load-bearing capacity are given in NAVFAC DM-21.07.

11.2.1.4 Functional Condition. The functional condition (friction, foreign object damage, profile, surface drainage, etc.) of an airfield pavement is just as important as its structural condition. The functional condition of the pavement is determined by its surface characteristics, and determines the ride quality and safety of the pavement. Functional overlays are discussed later in this section.

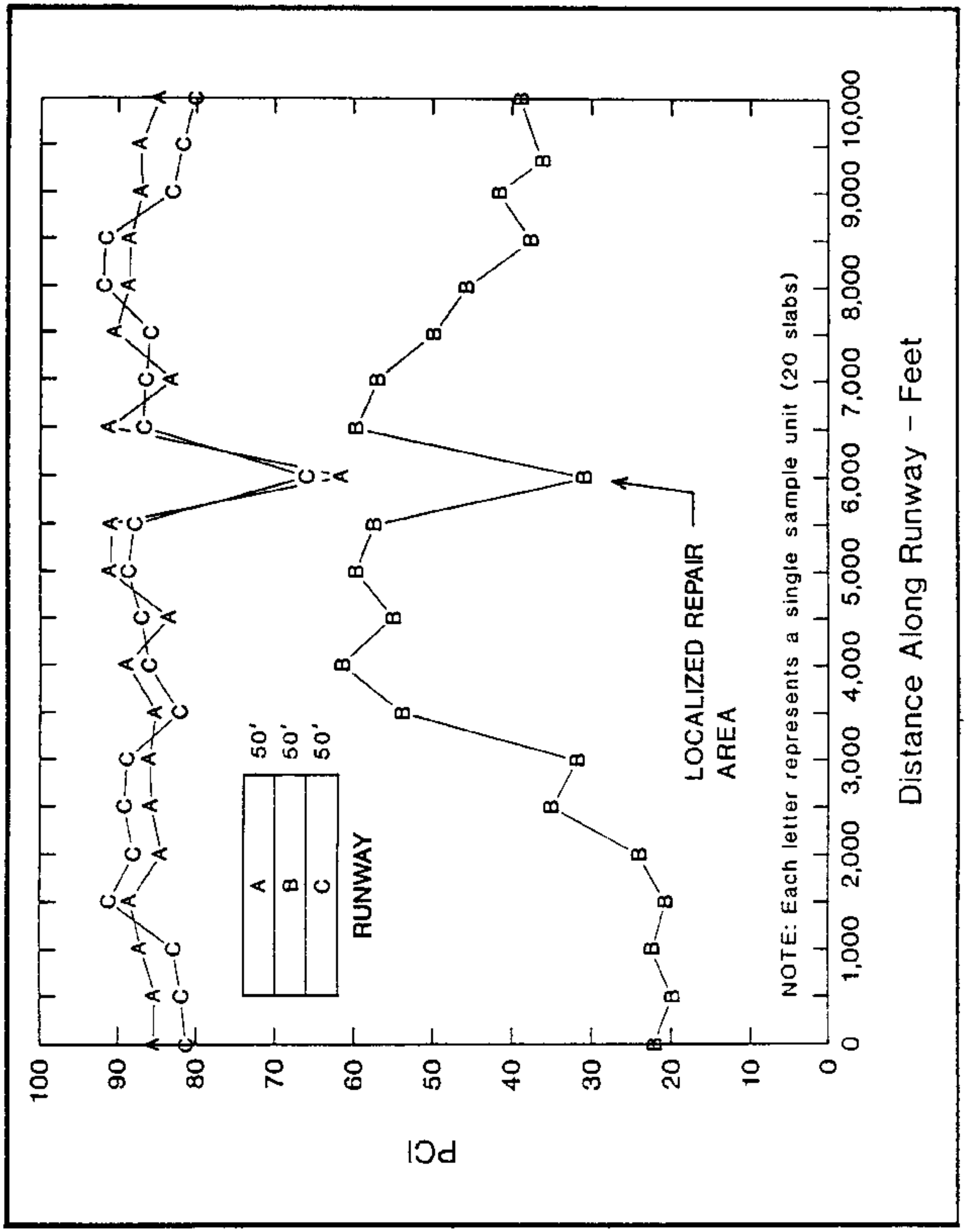


Figure 20
Example PCI Profile for a Runway Center and Edges

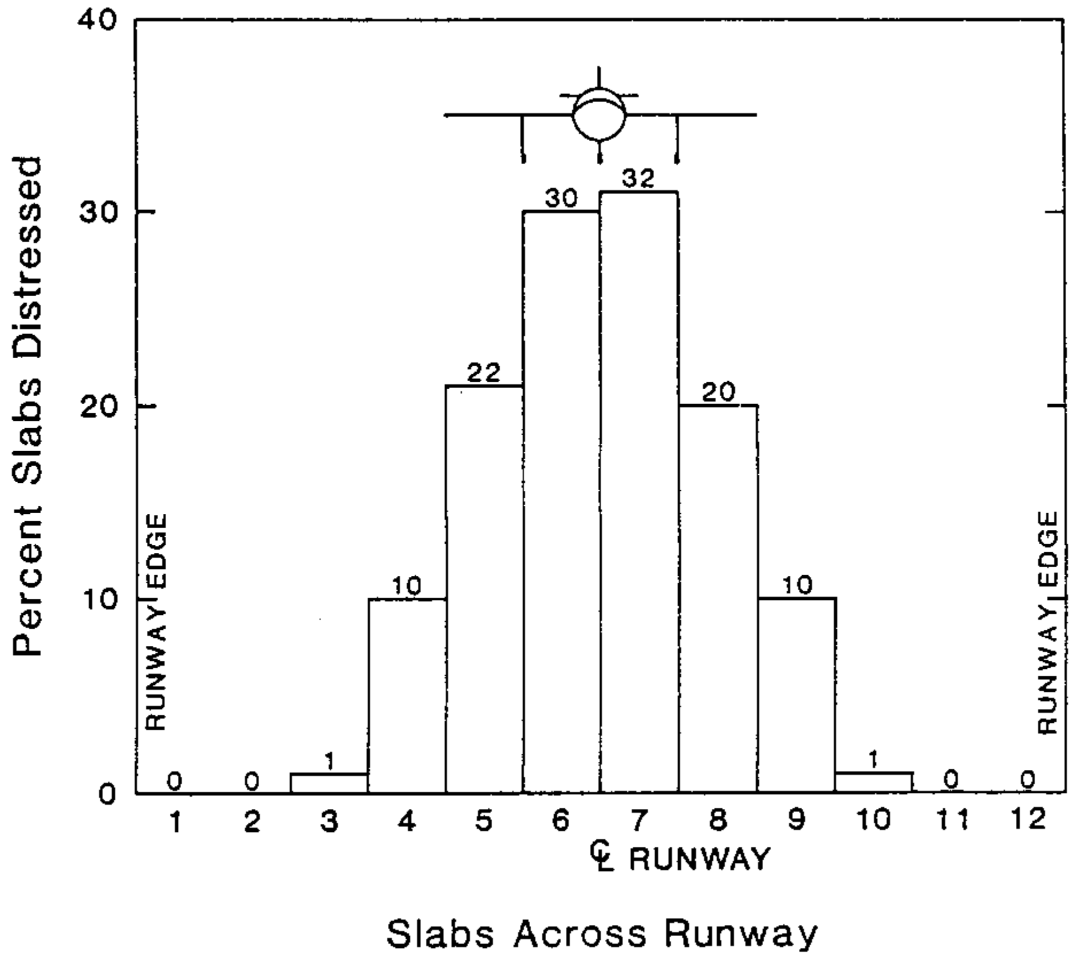


Figure 21
Example Distribution of Distressed Slabs Across Runway

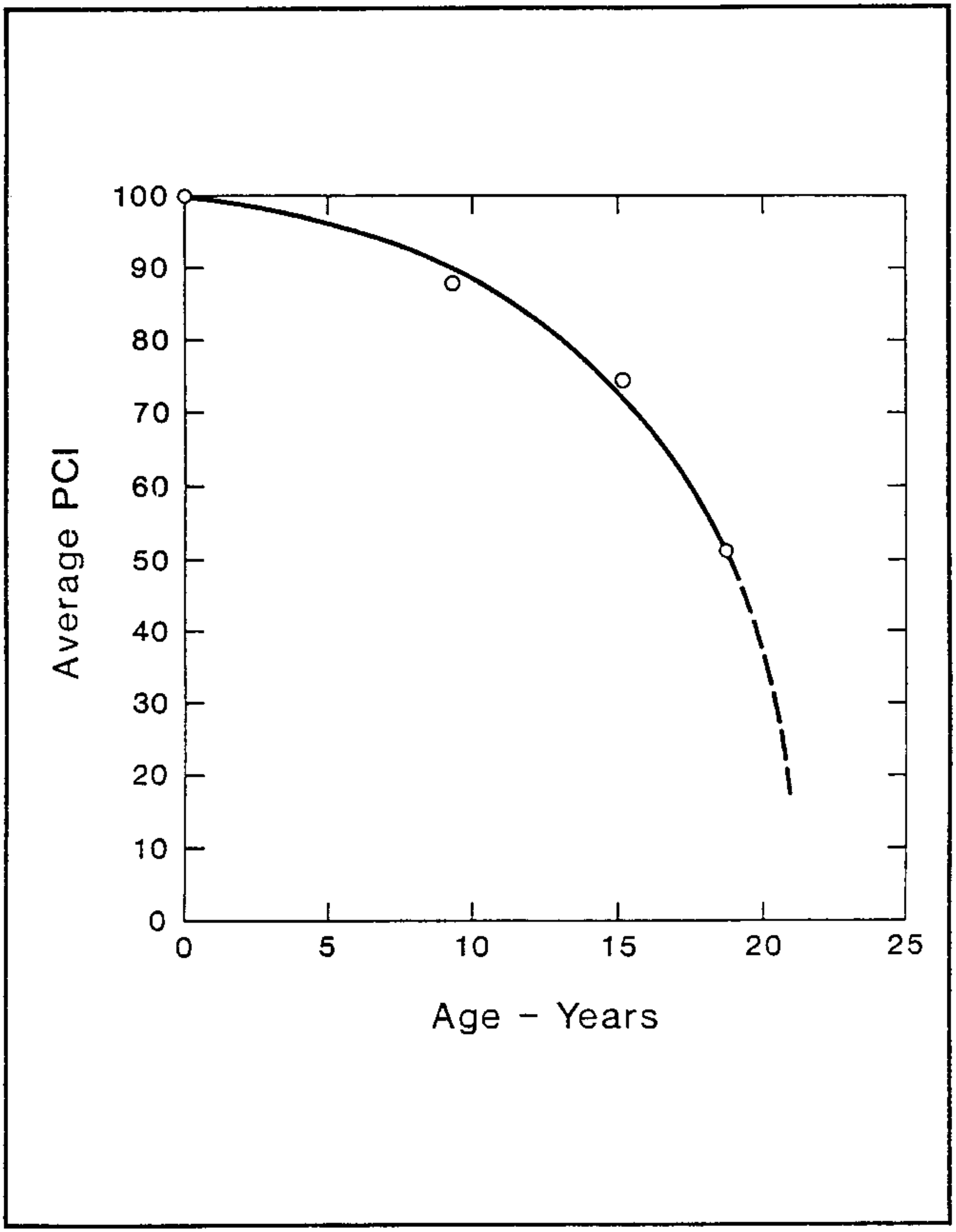


Figure 22
Example Plot of PCI Over Time for a Specific Pavement

a) Foreign Object Damage (FOD). Loose materials from spalls or scaling could damage jet aircraft engines. Take corrective action if loose materials from the pavement exist.

b) Runway Friction. The friction characteristics of an airfield runway pavement decrease steadily from the time of construction, increasing the potential for hydroplaning. The existence of rubber buildup, flat cross slopes, and absence of texture are all indicators of potential for hydroplaning. Procedures for measuring skid resistance are described in NAVFAC DM-21.09, Skid Resistant Runway Surfaces.

c) Runway Roughness. Pavement long wave roughness may be hazardous to aircraft. However, due to varying aircraft characteristics, different aircraft types may experience different amounts of "roughness" on the same pavement. The assessment of pavement roughness by pilots is a good source of information. In addition, roughness caused by localized spalling, faulting, etc., can cause tire damage.

11.2.1.5 Maintenance History. The extent and type of previous maintenance applied to the pavement is an indication of the types of distress which have existed in the pavement in the past, and thus might develop in the future unless addressed by the pavement rehabilitation alternative selected. Consult records of past maintenance and observe any visible previous maintenance on the pavement.

11.2.2 Need for Structural Improvement. Structural improvement of a pavement may be warranted for any of the following reasons:

a) a high rate of pavement deterioration,

b) a deficiency in load-bearing capacity,

c) distress which is predominantly load-related: transverse and longitudinal cracks, corner breaks, shattered slabs, certain types of joint spalls (e.g., keyway spalling), localized punchouts, and patching of load-related distress,

d) a high level of past maintenance,

e) a change in mission (aircraft types expected to use the facility) necessitating greater load-bearing capacity.

11.3 Data Collection for Overlay Design. The specific data items which are required inputs to the overlay thickness design procedure are described below.

11.3.1 Existing Slab Thickness. The thickness of the existing concrete slab may be available from original construction records. The as-built concrete slab thickness should be determined by taking cores.

11.3.2 Existing Concrete Flexural Strength. The flexural strength of the existing slab is used for the design of bonded concrete and for asphalt concrete overlays. It is determined by one of the following methods:

a) test beams sawed from the existing concrete slabs and tested in accordance with ASTM C78,

b) cores (must be 6 inches (150 mm) diameter) are cut and tested in splitting tensile strength in accordance with ASTM C496 and the flexural strength computed as:

EQUATION: $$FS = (1.02 \times ST) + 200 \qquad (4)$$

WHERE: FS = 3rd point flexural strength, psi
ST = splitting tensile strength, psi

c) Estimated from concrete strength data from construction records. Concrete strength may be measured by compressive strength tests but must be expressed in terms of flexural strength. An approximate value for the third-point flexural strength based on compressive strength is as follows:

$$7\sqrt{f'_c} \leq FS \leq 9\sqrt{f'_c}$$

The value determined for the flexural strength at the time of construction may be increased somewhat to account for the normal increase over time. The flexural strength for old concrete is typically 1.1 to 1.3 times the 28-day flexural strength at the time of construction.

11.3.3 Concrete Overlay Flexural Strength. The 28-day third-point loading flexural strength is used for design of partially bonded and unbonded concrete overlays. The design flexural strength should be as high as practicable and economical but not less than 650 pounds per square inch (4 482 000 Pa). See Section 8, paragraph 2.a. for further discussion.

11.3.4 Effective k Value. There are three procedures for determining the effective k value at the top of the base course for use in design:

a) Actual k value tests may have been conducted for previous pavement evaluation studies.

b) The standard method of determining the static k value beneath the existing slab is by plate bearing tests performed in accordance with ASTM D1196. Representative slabs are removed and the k value is measured at the top of the base. Any available soil borings should be considered in planning locations for the tests.

c) Estimation of subgrade k value using approximate correlations with soil types and densities, and then using Figure 2 to determine the effective k value at top of base.

d) Procedures for determining the k value through nondestructive deflection testing (NDT) at the top of the slab are under development.

11.3.5 Load Transfer Measurement. Perform nondestructive deflection testing to determine the average deflection load transfer of the transverse and longitudinal joints in the existing concrete slab. The loading plate is placed adjacent to the joint and the deflections are measured close to the joint an equal distance on each side of the joint. Deflection load transfer is defined as follows:

$$\text{Deflection Load Transfer} = \frac{\text{Deflection of Unloaded Slab X 100}}{\text{Deflection of Loaded Slab}}$$

The procedure to determine load transfer efficiency based on deflection measurements at the joints is under development and will be published in NAVFAC DM-21.07. In the absence of actual measured data, the following values are representative of those measured in the field:

Joint Type	Base Type	Representative Deflection Load Transfer, Percent
Weakened Plane	Granular	40 to 60
Weakened Plane	Stabilized	50 to 70
Keyway	Granular	50 to 70
Keyway	Stabilized	60 to 70
Doweled	Any type	70 to 90

11.3.6 Design Life and Design Traffic. The overlay thickness design procedure requires that the projected aircraft traffic using the facility over the design period be characterized by type and volume as described in Section 7. The minimum design life for Navy and Marine Corps facilities is 20 years.

11.4 Overlay Types. The two primary types of, overlays are flexible overlays constructed from asphalt materials and rigid overlays constructed from Portland cement concrete. These overlay types can be classified further as discussed in the following paragraphs.

11.4.1 Unbonded Concrete Overlay. An unbonded concrete overlay is used to increase the structural capacity of a pavement that is badly deteriorated. An asphalt concrete (AC) leveling course is constructed on top of the base slab to separate the overlay from the existing pavement and to provide a uniform surface for construction. This retards the reflection of joints and cracks from the base pavement into the overlay. The minimum allowable thickness for an unbonded concrete overlay is 8 inches (203 mm), except for basic training fields where 6-inch (150 mm) overlays may be used.

The minimum leveling course thickness is 1 inch (25 mm) and the maximum is 4 inches (100 mm). If a leveling course greater than 4 inches (100 mm) is required, the unbonded concrete overlay shall be designed as a new concrete pavement with an effective k value not greater than 500 psi (13 840 000 kg/m^3).

11.4.2 Bonded Concrete Overlay. Fully bonded concrete overlays are used only where the base slab is in good condition and no structural defects exist. The new concrete is placed directly on top of the base slab. Prior to placing the overlay, the surface of the base slab must be specially prepared to ensure a full bond with the overlay. By bonding the overlay to the existing base pavement, the new section behaves as a monolithic slab.

Because the new section behaves as one slab, the required thickness for bonded overlays is less than the required thickness for an unbonded overlay. The minimum thickness for a bonded concrete overlay is 3 inches (75 mm). Joints in the overlay must be matched by type (expansion matched with expansion, contraction with contraction) and location to those in the base slab.

11.4.3 Partially Bonded Concrete Overlay. A partially bonded concrete overlay results when fresh concrete is placed directly on the existing slabs. With this procedure, some degree of bonding due to cementing action and friction may be achieved between the overlay and the existing pavement. The minimum thickness for partially bonded concrete overlays is 6 inches (150 mm). Joints in the overlay must be matched by type (expansion matched with expansion, contraction with contraction) and location to those in the base slab. However, additional joints may be placed in the overlay to control curling stresses (particularly where joints in the existing slab are greater than 15 ft. (4.6 m)).

11.4.4 Asphalt Concrete Overlay. Asphalt concrete overlays are commonly used to improve both the structural and functional characteristics of the existing pavement. The minimum thickness for structural asphalt concrete overlays on concrete pavement is 4 inches (100 mm). To improve rideability and to correct surface deficiencies, an overlay as thin as 2 inches may be used. Joints can be expected to reflect through the overlay in a short time.

11.4.5 Reinforced Concrete Overlays. Reinforced concrete overlays have been used successfully as alternatives to conventional non-reinforced concrete pavement overlays, particularly under unusual conditions, such as to reduce thickness due to grade and drainage problems. The following sections briefly discuss the major types of reinforced concrete overlays. Reinforced concrete overlay designs should be considered on a case-by-case basis and must be approved by the Engineering Field Division of the Naval Facilities Engineering Command.

11.4.5.1 Jointed Reinforced. An unbonded or partially bonded jointed reinforced concrete overlay may be used to strengthen an existing jointed non-reinforced or jointed reinforced concrete pavement. The steel reinforcement holds cracks in the overlay tightly closed to improve load transfer. See AFM 88-6 for additional information and design procedures on jointed reinforced concrete overlay design.

11.4.5.2 Continuously Reinforced. An unbonded continuously reinforced concrete overlay may be used to strengthen either an existing jointed non-reinforced or jointed reinforced concrete pavement. The major advantages of using a continuously reinforced overlay are: (a) the elimination of the need to match joints of the overlay with those of the original construction, (b) reduced impact of reflection cracks since they are held tightly closed, and (c) restoration of a smooth riding surface with no faulted or deteriorated joints or warped slabs. See AFM 88-6 for additional information on continuously reinforced overlay design.

11.4.5.3 Steel Fiber Reinforced. This paragragh is for interim guidance and will be revised at a later date. Steel fiber reinforced concrete (SFRC) overlays may be considered for applications requiring either an unbonded or partially bonded overlay. Because the initial cost of SFRC is higher than non-reinforced concrete, and because the long term performance of SFRC is relatively unknown, its use should be restricted only to limited special situations where a thinner than normal overlay is required. Use of SFRC must be approved by the Naval Facilities Engineering Command Headquarters.

a) Thickness Design. Design unbonded or partially bonded SFRC overlays using Equation (5). The thickness of the overlay may be based on the flexural strength achieved with SFRC. Design 28-day flexural strength should be in the range of 800 to 1000 psi (5.5 to 6.9 MPa).

b) Joint Design. For partially bonded SFRC overlays the joints in the overlay must match the type and location of the joints in the base pavement. See paragraph 8. For unbonded overlays the joint design criteria given in Section 9 applies. Joint spacing for unbonded SFRC overlays must not exceed that specified in Section 9 for non-reinforced concrete pavements. To assure that contraction joints will function, the saw cut must be at least one-third of the slab thickness.

c) Construction. The most common construction problems experienced with SFRC mixes are "clumping" of the fibers during batching and accumulations of loose surface fiber during the finishing operations. To avoid clumping the steel fibers must be introduced into the concrete mix as a ribbon feed through a 4-inch (102 mm) mesh screen or through a hopper to the fine aggregate conveyor belt of a central mix plant. During mixing, fibers must not be allowed to stack up while being introduced.

Some finishing operations may bring an excess of fibers to the surface or may tear the surface. To avoid accumulations of loose surface fiber the following finishing operations are necessary:

a) Strike off with a vibrating metal screed.

b) Use magnesium floats for smoothing. Do not use woo floats.

c) Use a hand-operated cylindrical grid-type roller commonly called a "rollerbug" to depress the steel fiber beneath the surface of the pavement.

d) For texture apply a light transverse broom. Do not use a burlap drag.

Additional guidance on batching and placing SFRC pavements can be found in the American Concrete Institute Committee Report No. AC1 544.3R, Specifying, Mixing, Placing and Finishing Steel Fiber Reinforced Concrete.

11.4.6 Selection of Overlay Type. Base the selection of the overlay type on an analysis of the information gathered during the project survey and evaluation.

The structural adequacy of the pavement must be determined. If it is structurally adequate for the projected traffic or change in mission then no structural overlay is required; a functional overlay may be required based on an evaluation of the existing friction characteristics, foreign object damage (FOD), pavement surface profile, and roughness. If the pavement is structurally deficient, then a structural overlay will be required.

The distress in the existing pavement must also be considered in selecting the overlay design type. Where surface distress is minimal, any economical type of overlay may be considered. Where distress is advanced, the alternatives may be limited to an unbonded concrete overlay or asphalt concrete overlay.

The amount of pre-overlay repair should also be considered. A pavement with advanced distress may be a candidate for a bonded or partially bonded concrete overlay only if extensive repair is first performed.

Consider initial cost, life-cycle cost, future maintenance, material availability, skilled construction availability, construction time, and energy and environmental constraints. General recommendations for the feasibility of each type of overlay are given in Figure 23.

11.5 <u>Pre-Overlay Repair</u>. Consider the amount of pre-overlay repair needed on the existing pavement. This amount is a function of the type of overlay, the structural adequacy of the existing pavement, the distress types and severities present in the existing pavement, the thickness of the overlay, and overall cost considerations. Also consider reflective cracking when designing asphalt concrete overlays.

Place all overlays, except unbonded concrete overlays, only on pavements that have a PCISTR 2 35. PCI_{STR} is defined in paragraph 7 of this section. Pavements with an existing $PCI_{STR} < 35$ must be repaired to raise the PCI_{STR} to $\geq$ 35 before the overlay is placed.

11.5.1 <u>Localized Repair</u>. Many pavements have areas of localized distress caused by joint deterioration, construction errors, or materials and soil variability. Prior to overlaying the pavement, identify the existing distress and apply treatments which both repair the existing distress and prevent, as much as possible, its recurrence. In concrete pavements, repair shattered slabs, corner breaks, broken portions of slabs, edge and corner voids (or loss of support), and joint spalling prior to the overlay.

Match the amount of pre-overlay repair to the type of overlay being applied. A greater amount of the existing distress will need to be repaired for a fully bonded concrete overlay than for an unbonded concrete overlay.

11.5.2 <u>Bonded and Partially Bonded Concrete Overlays</u>. Bonded concrete overlays should be used only when the existing rigid pavement is in good condition or where all serious distress has been repaired. Remove and replace all badly cracked and broken slabs (three or more pieces). Seal cracks and joints l/2 inch (12 mm) or more in width prior to the overlay. Undersea1 all areas of localized voids and pumping to restore support to the existing slabs.

11.5.3 <u>Unbonded Concrete Overlay</u>. Unbonded concrete overlays require the least amount of pre-overlay repair and may be used where severe structural defects are present. However, as the amount of structural distress increases, the required overlay thickness increases. Evaluate the effect that full depth repair of structural defects will have on the required overlay thickness and costs.

PCI RATING	TYPE OF OVERLAY			
	BONDED CONCRETE	PARTIALLY BONDED CONCRETE	UNBONDED CONCRETE	ASPHALT CONCRETE
EXCELLENT	YES	NO (costly)	NO (costly)	NO (not normally required)
VERY GOOD	YES	YES	NO (costly)	NO (not normally required)
GOOD	YES (w/repair)	YES	NO (costly)	YES
FAIR	NO	YES (w/ repair)	YES	YES (w/repair)
POOR	NO	NO	YES	YES (w/repair)
VERY POOR	NO	NO	YES	NO
FAILED	NO	NO	YES (w/repair)	NO

Figure 23
General Guide for Type of Overlay Selection

11.4.5 Reinforced Concrete Overlays. Reinforced concrete overlays have been used successfully as alternatives to conventional non-reinforced concrete pavement overlays, particularly under unusual conditions, such as to reduce thickness due to grade and drainage problems. The following sections briefly discuss the major types of reinforced concrete overlays. Reinforced concrete overlay designs should be considered on a case-by-case basis and must be approved by the Engineering Field Division of the Naval Facilities Engineering Command.

11.4.5.1 Jointed Reinforced. An unbonded or partially bonded jointed reinforced concrete overlay may be used to strengthen an existing jointed non-reinforced or jointed reinforced concrete pavement. The steel reinforcement holds cracks in the overlay tightly closed to improve load transfer. See AFM 88-6 for additional information and design procedures on jointed reinforced concrete overlay design.

11.4.5.2 Continuously Reinforced. An unbonded continuously reinforced concrete overlay may be used to strengthen either an existing jointed non-reinforced or jointed reinforced concrete pavement. The major advantages of using a continuously reinforced overlay are: (a) the elimination of the need to match joints of the overlay with those of the original construction, (b) reduced impact of reflection cracks since they are held tightly closed, and (c) restoration of a smooth riding surface with no faulted or deteriorated joints or warped slabs. See AFM 88-6 for additional information on continuously reinforced overlay design.

11.4.5.3 Steel Fiber Reinforced. This paragragh is for interim guidance and will be revised at a later date. Steel fiber reinforced concrete (SFRC) overlays may be considered for applications requiring either an unbonded or partially bonded overlay. Because the initial cost of SFRC is higher than non-reinforced concrete, and because the long term performance of SFRC is relatively unknown, its use should be restricted only to limited special situations where a thinner than normal overlay is required. Use of SFRC must be approved by the Naval Facilities Engineering Command Headquarters.

a) Thickness Design. Design unbonded or partially bonded SFRC overlays using Equation (5). The thickness of the overlay may be based on the flexural strength achieved with SFRC. Design 28-day flexural strength should be in the range of 800 to 1000 psi (5.5 to 6.9 HPa).

b) Joint Design. For partially bonded SFRC overlays the joints in the overlay must match the type and location of the joints in the base pavement. See paragraph 8. For unbonded overlays the joint design criteria given in Section 9 applies. Joint spacing for unbonded SFRC overlays must not exceed that specified in Section 9 for non-reinforced concrete pavements. To assure that contraction joints will function, the saw cut must be at least one-third of the slab thickness.

11.6.4 Asphalt Concrete Overlay. No special surface preparations are required prior to placing an asphalt concrete overlay. Prior to placing the asphalt concrete overlay, sweep the existing surface clean of all dust, dirt, and foreign matter, and place a tack coat. If the existing pavement is rough due to slab distortion, faulting, or settlement, place a leveling course prior to the overlay.

11.7 Overlay Thickness Design. The overlay thickness design equations are empirical equations originally based on field test sections and modified based on in-service airfield pavements. The general form of the equation for Portland cement concrete overlays on rigid pavement is given in Equation (5). The general form of the equation for asphalt concrete overlays on rigid pavement is given in Equation (6).

EQUATION:

$$T_o = \sqrt[p]{(T_n)^p - C_r(T_e)^p} \qquad (5)$$

WHERE:

T_o = required thickness of concrete overlay, inches
T_n = required single slab thickness for a new design, inches
T_e = thickness of existing concrete slab, inches
C_r = condition factor for existing rigid pavement ranging from 0.35 to 1.0
p = 1.0 for bonded overlay
1.4 for partially bonded overlay
2.0 for unbonded overlay

EQUATION:

$$T_o = 2.5\,(FT_n - C_bT_e) \qquad (6)$$

WHERE:

T_o = required thickness of asphalt overlay, inches
T_n = required single slab thickness for a new design, inches
T_e = thickness of existing concrete slab, inches
C_b = condition factor for existing rigid pavement ranging from 0.50 to 1.0
F = factor which controls the degree of cracking in the existing rigid pavement

11.7.1 Concrete Overlay Thickness Design. The required concrete overlay thickness is obtained using Equation (5). The required single slab thickness for a new design is obtained using the thickness design procedures in Section 8 for the projected future traffic. The existing effective k value is obtained as described in paragraph 3.d. (Section 11). The flexural strength used for design is either that of the existing concrete slab or that of the new concrete overlay:

Type of Overlay	Design Flexural Strength
Bonded concrete	Existing slab
Asphalt concrete	Existing slab
Partially bonded concrete	New overlay
Unbonded concrete	New overlay

The condition factor, C,, is determined from a relationship based on the measured PCI. The appropriate exponent in Equation (5) is selected based on the type of overlay being designed.

11.7.1.1 Required Inputs. Complete the heading to Table 22 by recording the pavement identification and traffic area (primary/channelized or secondary/unchannelized), the trial single slab thickness, the base effective k, the design flexural strength, the type of overlay being designed, and the load transfer efficiency (percent) in the existing slab.

Each aircraft type and its design gear load is entered in Columns 1 and 2. The forecasted number of passes (departures) of each aircraft over the design period is entered in Column 3. The forecasted number of passes of each aircraft is then converted to number of coverages by dividing by the appropriate pass-coverage ratio for each aircraft from Table 14 in Section 7 of this manual.

11.7.1.2 Determination of Flexural Stress. Select a trial single slab thickness and record it in the space provided for the iteration being performed. The interior flexural stress is determined as described in Section 8 of this manual. Enter the appropriate Figure 9 through 13 with the trial slab thickness, effective k value, and gear load to determine the interior flexural stress for each aircraft.

If a bonded or partially bonded concrete overlay is being designed (with matching joints), an adjustment must be made to the interior flexural stress recorded in Column 6 to account for the actual load transfer in the existing joints. Enter Figure 24 with the existing joint percent load transfer and determine the load transfer adjustment factor for each type of aircraft. The load transfer adjustment factor is recorded in Column 7. The flexural interior stress in Column 6 is then multiplied by the load transfer adjustment factor in Column 7 to obtain the critical edge stress which is recorded in Column 8.

Table 22
Fatigue Damage Summary Sheet for Overlay Design

PAVEMENT IDENTIFICATION ______ TRAFFIC AREA ______
NEW SLAB THICKNESS: ______ SINGLE WHEEL AIRCRAFT TIRE PRESSURE, psi
BASE EFFECTIVE K: ______ 1. ______ 1. ______
DESIGN FS: ______ 2. ______ 2. ______
TYPE OF OVERLAY: ______ LOAD TRANSFER EFFICIENCY: ______

AIRCRAFT 1	DESIGN GEAR LOAD 2	PROJECTED PASSES 3	P/C 4	PROJECTED COVERAGES (n) 5	INTERIOR STRESS 6	LOAD TRANSFER ADJUSTMENT FACTOR 7	CRITICAL STRESS 8	CRITICAL STRESS/ FS 9	ALLOWABLE COVERAGES (N) 10	FATIGUE LIFE CONSUMED (n/N) 11
									$\Sigma n/N=$	

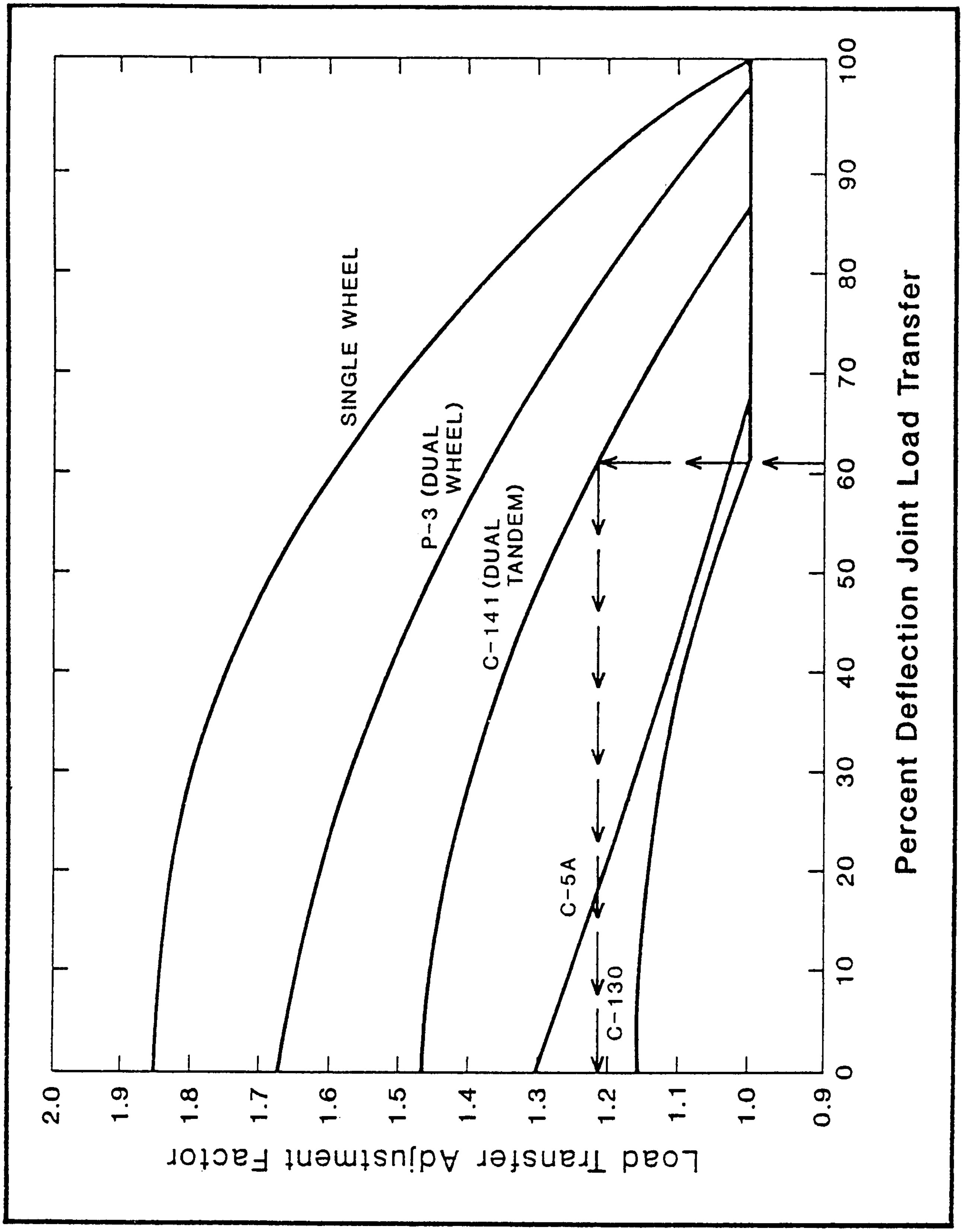

SINGLE WHEEL
P-3 (DUAL WHEEL)
C-141 (DUAL TANDEM)
C-5A
C-130
Load Transfer Adjustment Factor
2.0
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
0.9
0
10
20
30
40
50
60
70
80
90
100
Percent Deflection Joint Load Transfer

Figure 24

If an unbonded concrete overlay is being designed, the flexural stress does not need to be adjusted for the load transfer efficiency of the existing slabs unless the joints in the overlay will be matched with the joints in the existing slab. For mismatched joints, the load transfer adjustment factor is one (1.0) and the critical stress is recorded in Column 8 as the same value as the interior stress from Column 6.

11.7.1.3 Fatigue Life Consumption. The fatigue life consumption is determined as described in Section 8 of this manual. The fatigue life consumed must be less than 1.0 (or 100 percent). The process of selecting a slab thickness, determining the flexural stress, and calculating the fatigue life consumption is repeated until the slab thickness which corresponds to an acceptable value for Miner's damage (less than 1.00 or 100 percent) is determined.

11.7.1.4 Determination of CÚr¿. The condition of the existing concrete slab is taken into consideration during the design of the concrete overlay. The existing concrete pavement may have suffered deterioration from climate and material durability as well as fatigue damage from traffic loadings throughout the life of the pavement. To account for these factors, a condition factor, CÚr¿, is used to evaluate the structural condition of the existing concrete slab. The condition factor, CÚr¿, is determined using the results of the PCI survey. When determining CÚr¿ the only distresses considered are those associated with structural loading. These include:

a) longitudinal, transverse, and diagonal cracks of medium to high severity,
b) corner breaks of any severity,
c) all large patches of load-associated failures,
d) pumping,
e) settlement or faulting of any severity,
f) shattered slabs of any severity,
g)certain types of joint spalls believed to be load-associated (e.g., keyway failures).

The PCIÚSTR¿ (PCI based on structural distress only) is then calculated using only these structural distresses.

The value of CÚr¿ ranges from 0.35 to 1.0. A CÚr¿ of 0.35 corresponds to a condition where approximately 60 percent of the concrete slabs are shattered with a severity level of medium, or 50 percent of the slabs have high severity longitudinal, transverse, or diagonal cracks. A CÚr¿ of 1.0 corresponds to a condition where the pavement is in very good condition with little or no structural cracking.

A correlation between PCI computed from structural distress only (PCI_{STR}) and C_r is shown in Figure 25.

11.7.1.5 Determination of Overlay Thickness (T_o). The required thickness of the concrete overlay (T_o) can be computed using Equation (5) and the appropriate exponent for the type of overlay. An exponent of 1.0 should be used for bonded concrete overlays, 1.4 for partially bonded concrete overlays, and 2.0 for unbonded overlays. The calculated thickness (T_o) is rounded to the nearest whole inch to obtain the design concrete overlay thickness. If the computed overlay thickness is less than or equal to 0.25 inches (6 mm) greater than a whole inch, the thickness is rounded downward (e.g., 10.15 inches is rounded to 10 inches). If the thickness is more than 0.25 inches (6 mm) greater than a whole inch, the thickness is rounded upward (e.g., 10.30 inches is rounded to 11 inches).

11.7.2 Asphalt Concrete Overlay Thickness Design. The required asphalt concrete overlay thickness is obtained using Equation (6). The procedure described in sub-paragraph 7.a. above should first be used to determine the critical flexural stress and the required single slab thickness (T_n) for a new bonded concrete overlay design. Then this thickness is adjusted as described below to obtain the required thickness of asphalt overlay (T_o).

11.7.2.1 Determination of C_b. The structural condition of the existing concrete slab is taken into consideration by use of a condition factor, Cb. This factor ranges from 0.50 to 1.0. A C_b value of 1.0 indicates that the existing slabs are in excellent condition and contain none or only nominal cracking. A C_b value of 0.50 indicates that the existing slabs contain multiple cracking.

A relationship between PCI_{STR} and C_b is shown in Figure 26. PCI_{STR} is computed from the existing structural distress in exactly the same manner as described for the rigid pavement condition factor.

11.7.2.2 Determination of F. The required single slab thickness for a new design (T_n) is modified by a factor "F" which controls the amount of cracking which is allowed to occur in the base concrete slab. The "F" factor essentially reduces the required single slab thickness which will increase the critical stress. This reduction in thickness is allowed because an asphalt overlay is allowed to crack and deflect more than a rigid pavement. More cracking is allowed in an asphalt overlay because the asphalt overlay can conform to greater deflections than a rigid pavement without excessive spalling.

The "F" factor decreases as the effective k value on top of the base course increases. Thus, as the effective k value increases, the thickness of the asphalt concrete overlay decreases. This reduction in thickness is allowed because the increased slab support provides a stable foundation that reduces movements and deflections when the existing concrete slab begins to crack. Determine the appropriate "F" factor from Figure 27.

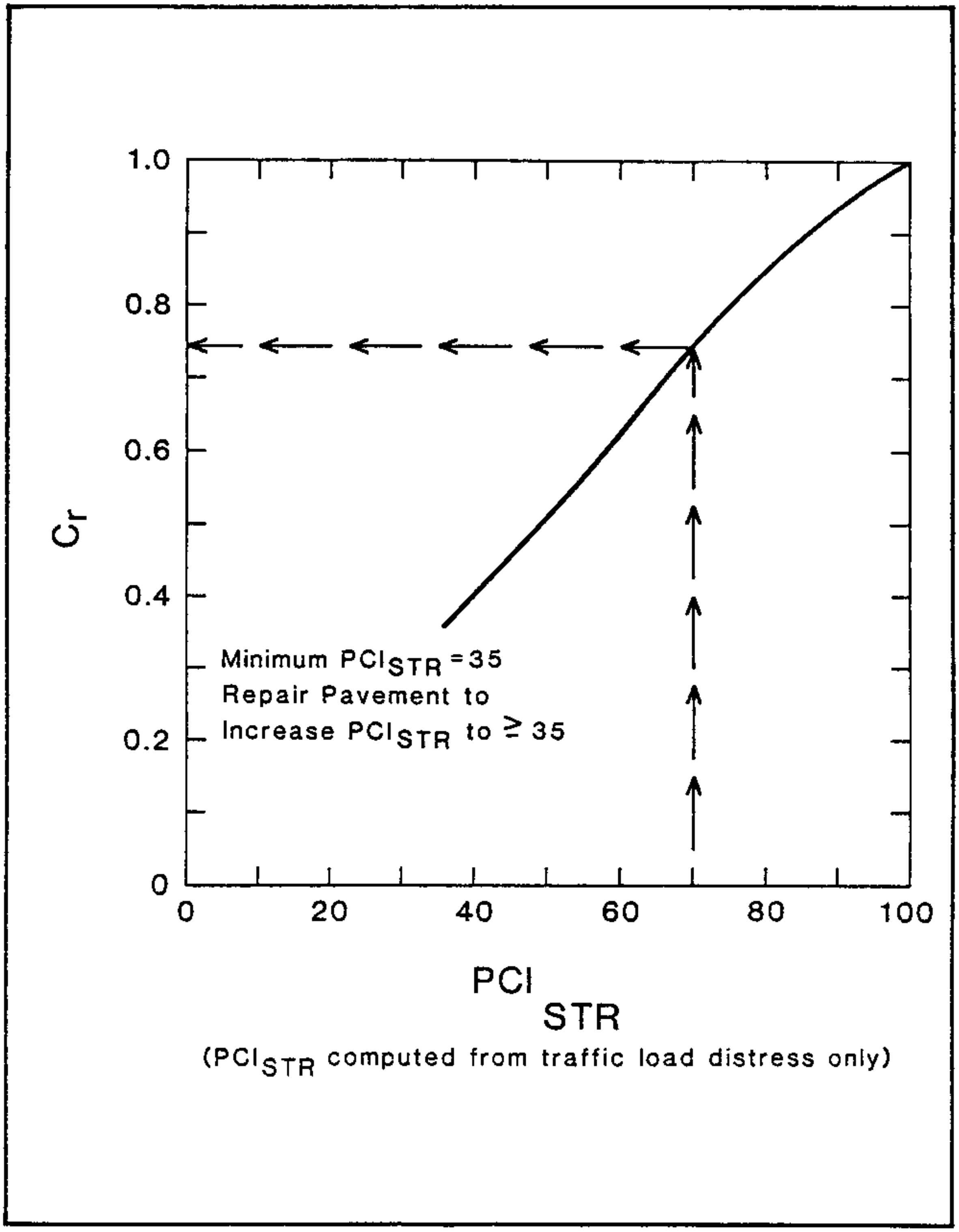


Figure 25
Chart for Determining C_r (for Rigid Overlays)

11.7.2.3 Determination of Asphalt Concrete Overlay Thickness. Determine the required thickness of the asphalt concrete overlay (T_o) using Equation (6). The constant 2.5 is the equivalency factor that converts concrete thickness to asphalt concrete thickness. The calculated thickness (T_o) is rounded up to the nearest inch to obtain the design asphalt concrete overlay thickness. If the computed overlay thickness is less than or equal to 0.25 inches (6 mm) greater than a whole inch, the thickness is rounded downward (e.g., 4.2 inches is rounded to 4 inches). If the thickness is more than 0.25 inches (6 mm> greater than a whole inch, the thickness is rounded upward (e.g., 4.4 inches is rounded to 5 inches).

If the asphalt concrete overlay thickness exceeds the thickness of the base concrete slab, the designer should consider designing the overlay as a flexible pavement and treating the existing rigid pavement as a high quality base material.

11.8 Joint Design. The joint design for a concrete overlay on a rigid pavement is governed by the type of overlay (i.e., bonded, partially bonded, or unbended) and the joint design of the existing slabs. Design joints for concrete overlays in accordance with Section 9 of this manual.

11.8.1 Bonded and Partially Bonded Concrete Overlays. For all bonded and partially bonded concrete overlays, the location of the joints in the overlay must exactly match all the joints in the existing rigid pavement. The joints in the overlay do not have to be the same type as the joints in the existing rigid pavement, except that all expansion joints in the base pavement must be matched with expansion joints in the overlay. Sawed joints for bonded concrete overlays must be sawed completely through the overlay.

11.8.2 Unbonded Concrete Overlay. For unbonded concrete overlays, the joints in the overlay do not have to match the joints in the existing rigid pavement. In fact, a complete mismatch of joints should be considered to improve load transfer of the overlay joints.

11.8.3 Asphalt Concrete Overlay. The paving lanes for the asphalt concrete overlay should be laid out to prevent construction joints in the overlay from coinciding with joints in the existing rigid pavement. Construction joints in successive layers of the asphalt overlay should also be offset.

11.9 Overlay Design Examples. Detailed design examples follow to illustrate the steps to determine the required thickness for a concrete or asphalt concrete overlay.

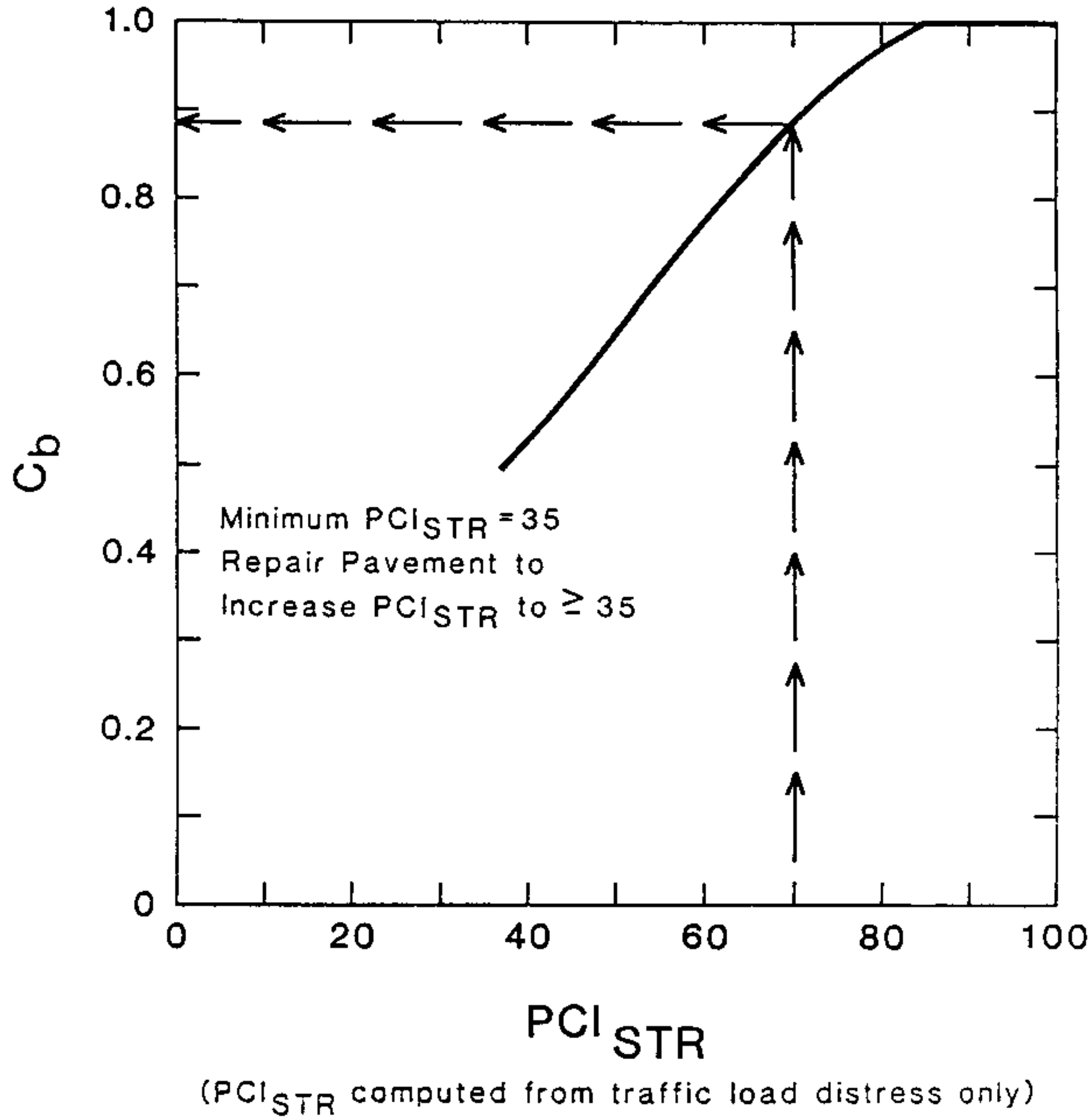


Figure 26
Chart for Determining C_b (for Flexible Overlays)

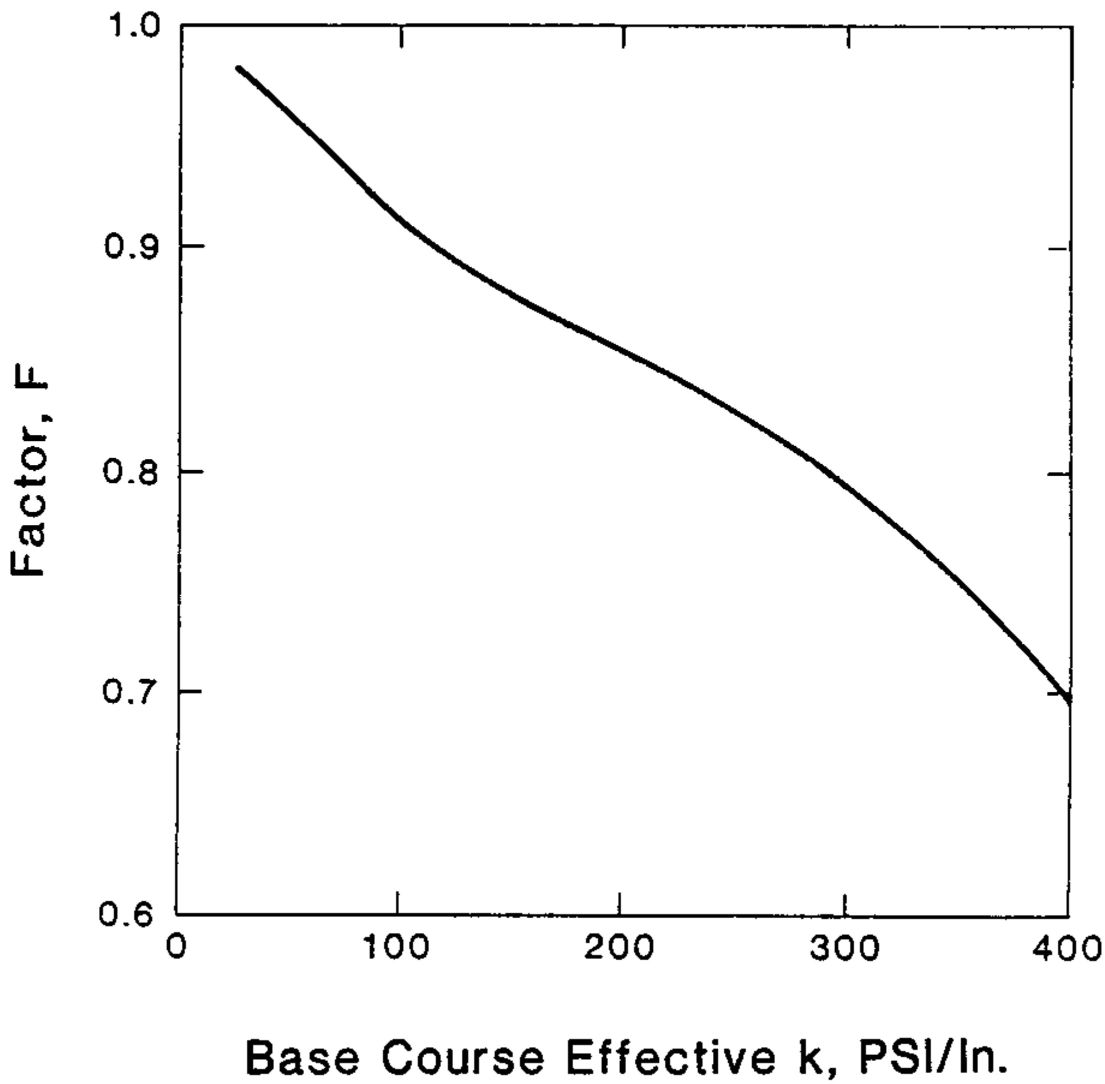


Figure 27
Chart for Determining F Factor for Design of Flexible Pavements

11.9.1 Concrete Overlay Design Example. A concrete overlay is to be designed for an existing runway to accommodate a change in mission. The overlay is to be designed for the following traffic expected to use the facility over the next 20 years:

Aircraft	Passes, 20 Years	Design Gear Load Pounds	
c-141	25,000	155,000	(7EiO)
c-130	50,000	84,000	(38 000)
C-5A	25,000	190,000	(86 000)
P-3	50,000	68,000	(31 000)
F-14	100,000	30,000	(13 600)

The following information was obtained during the pavement evaluation:

Existing concrete slab thickness = 9.0 inches
Average joint deflection load transfer = 60 percent
Effective k value = 250 psi (on top of existing base course)
Flexural strength of existing concrete slab = 750 psi
Flexural strength of new concrete overlay = 750 psi
PCI_{STR} = 70 for interior keel portion,
= 45 for runway ends keel section (within 1000 feet of each end of runway)

Apply extensive pre-overlay repair consisting of full depth slab replacement of badly cracked and shattered slabs and subsealing in areas where loss of support was detected to each runway end. The pre-overlay repair in these areas will raise the PCI_{STR} to 70. The overlay thickness for the entire runway can now be determined based on an existing PCI_{STR} of 70. The condition factor C_r is determined to be 0.74 from Figure 25.

The traffic projections, average percent load transfer, k value, and flexural strength of the existing concrete slab are used to determine the required single slab thickness. The load transfer adjustment factors were determined from Figure 24 for each aircraft using a measured load transfer efficiency of 60 percent.

Trial calculations to determine the required single slab thickness for a bonded concrete, or partially bonded concrete or asphalt concrete overlay for the (primary) channelized traffic areas (runway ends) are summarized in Table 23. Trial calculations to determine the single slab thickness for the (secondary) unchannelized traffic areas (runway interior) are also performed (but not shown).

Trial calculations to determine the required single slab thickness for an unbonded overlay for primary traffic areas are summarized in Table 24. Calculations for secondary traffic areas are also performed. The required single slab thicknesses (T_n) are summarized below:

Table 23
Design Example for Single Slab Thickness for Bonded or Partially Bonded Concrete Overlay

PAVEMENT IDENTIFICATION: **RUNWAY 4-22** TRAFFIC AREA: **CHANNELIZED**
NEW SLAB THICKNESS: **13.5"** SINGLE WHEEL AIRCRAFT TIRE PRESSURE, psi
BASE EFFECTIVE K: **250 PCI** 1. **F-14** 1. **240**
DESIGN FS: **750 PSI** 2. 2.
TYPE OF OVERLAY: **BONDED, PARTIALLY BONDED, or AC** LOAD TRANSFER EFFICIENCY: **60%**

AIRCRAFT 1	DESIGN GEAR LOAD 2	PROJECTED PASSES 3	P/C 4	PROJECTED COVERAGES (n) 5	INTERIOR STRESS 6	LOAD TRANSFER ADJUSTMENT FACTOR 7	CRITICAL STRESS 8	CRITICAL STRESS/ FS 9	ALLOWABLE COVERAGES (N) 10	FATIGUE LIFE CONSUMED (n/N) 11
C-141	155,000	25,000	3.49	7,163	390	1.22	476	0.63	14,000	0.51
C-130	84,000	50,000	4.36	11,468	300	1.00	300	0.40	UNLIMITED	-
C-5A	190,000	25,000	1.62	15,432	330	1.03	340	0.45	2,300,000	0.01
P-3	68,000	50,000	3.45	14,493	330	1.37	452	0.60	32,000	0.45
F-14	30,000	100,000	8.58	11,655	230	1.58	363	0.49	720,000	0.02
									$\Sigma n/N =$	0.99

Table 23 (Continued)
Design Example for Single Slab Thickness for Bonded or Partially Bonded Concrete Overlay

PAVEMENT IDENTIFICATION: RUNWAY 4-22 — TRAFFIC AREA: CHANNELIZED

NEW SLAB THICKNESS: 13.0"

BASE EFFECTIVE K: 250 PCI

DESIGN FS: 750 PSI

TYPE OF OVERLAY: BONDED, PARTIALLY BONDED, or AC

SINGLE WHEEL AIRCRAFT: 1. F-14 2. ___

TIRE PRESSURE, psi: 1. 240 2. ___

LOAD TRANSFER EFFICIENCY: 60%

AIRCRAFT 1	DESIGN GEAR LOAD 2	PROJECTED PASSES 3	P/C 4	PROJECTED COVERAGES (n) 5	INTERIOR STRESS 6	LOAD TRANSFER ADJUSTMENT FACTOR 7	CRITICAL STRESS 8	CRITICAL STRESS/ FS 9	ALLOWABLE COVERAGES (N) 10	FATIGUE LIFE CONSUMED (n/N) 11
C-141	155,000	25,000	3.49	7,163	410	1.22	500	0.67	4,500	1.59
C-130	84,000	50,000	4.36	11,468	320	1.00	320	0.43	UNLIMITED	-
C-5A	190,000	25,000	1.62	15,432	340	1.03	350	0.47	1,300,000	0.01
P-3	68,000	50,000	3.45	14,493	355	1.37	486	0.65	8,000	1.81
F-14	30,000	100,000	8.58	11,655	250	1.58	395	0.53	240,000	0.05
									$\Sigma n/N =$	3.46

Traffic Area	Required Single Slab Thickness, T_n - inches Fully Bonded	Partially Bonded	Unbonded
Channelized	13.5 (345 mm)	13.5 (356 mm)	12.0 (305 mm)
Unchannelized	13.5 (345 mm)	13.5 (345 mm)	11.5 (290 mm)

Note: For this example, a slight change in slab thickness results in a large change in damage. Thus, there is no difference in thickness between primary and secondary areas.

The required concrete overlay thickness (T_o), is determined as follows for a partially bonded overlay in the primary traffic area:

$$T_o = \sqrt[p]{(T_n)^p - C_r(T_e)^p}$$

$$T_o = \sqrt[1.4]{(13.5)^{1.4} - (0.74)*(9.0)^{1.4}}$$

T_o = 9.2 inches

Similar calculations can be repeated for unbonded and fully bonded overlays. The required concrete overlay thicknesses for each of these conditions are summarized below:

Summary of Required Concrete Overlay Thicknesses, T_o, for Design Example

Traffic Area	Fully Bonded	Overlay Thickness, T_o - inches Partially Bonded	Unbonded
Channelized	6.8 (175 mm)	9.2 (230 mm)	9.2 (230 mm)
Recommended:	7.0 (178 mm)	9.0 (229 mm)	9.0 (229 mm)
Unchannelized	6.8 (175 mm)	9.2 (230 mm)	8.5 (215 mm)
Recommended:	7.0 (178 mm)	9.0 (229 mm)	9.0 (229 mm)

The recommended design concrete overlay thickness is obtained by rounding the required overlay thickness to the whole inch as specified in Section 11, paragraph 7. Joint design for the unbonded concrete overlay should be in accordance with Section 9 of this manual.

Table 24
Design Example for Single Slab Thickness for Unbonded Concrete Overlay

PAVEMENT IDENTIFICATION **RUNWAY 4-22** TRAFFIC AREA **CHANNELIZED**
NEW SLAB THICKNESS: **11.5"** SINGLE WHEEL AIRCRAFT TIRE PRESSURE, psi
BASE EFFECTIVE K: **250 PCI** 1. **F-14** 1. **240**
DESIGN FS: **750 PSI** 2. 2.
TYPE OF OVERLAY: **UNBONDED** LOAD TRANSFER EFFICIENCY: **-**

AIRCRAFT 1	DESIGN GEAR LOAD 2	PROJECTED PASSES 3	P/C 4	PROJECTED COVERAGES (n) 5	INTERIOR STRESS 6	LOAD TRANSFER ADJUSTMENT FACTOR 7	CRITICAL STRESS 8	CRITICAL STRESS/ FS 9	ALLOWABLE COVERAGES (N) 10	FATIGUE LIFE CONSUMED (n/N) 11
C-141	155,000	25,000	3.49	7,163	490	1.0	490	0.65	8,000	0.90
C-130	84,000	50,000	4.36	11,468	385	1.0	385	0.51	400,000	0.03
C-5A	190,000	25,000	1.62	15,432	390	1.0	390	0.52	300,000	0.05
P-3	68,000	50,000	3.45	14,493	425	1.0	425	0.57	75,000	0.19
F-14	30,000	100,000	8.58	11,655	310	1.0	310	0.41	UNLIMITED	-
									$\Sigma n/N=$	1.17

Table 24 (Continued)
Design Example for Single Slab Thickness for Unbonded Concrete Overlay

PAVEMENT IDENTIFICATION **RUNWAY 4-22** TRAFFIC AREA **CHANNELIZED**
NEW SLAB THICKNESS: **12.0"** SINGLE WHEEL AIRCRAFT TIRE PRESSURE, psi
BASE EFFECTIVE K: **250 PCI** 1. **F-14** 1. **240**
DESIGN FS: **750 PSI** 2. 2.
TYPE OF OVERLAY: **UNBONDED** LOAD TRANSFER EFFICIENCY: **-**

AIRCRAFT 1	DESIGN GEAR LOAD 2	PROJECTED PASSES 3	P/C 4	PROJECTED COVERAGES (n) 5	INTERIOR STRESS 6	LOAD TRANSFER ADJUSTMENT FACTOR 7	CRITICAL STRESS 8	CRITICAL STRESS/FS 9	ALLOWABLE COVERAGES (N) 10	FATIGUE LIFE CONSUMED (n/N) 11
C-141	155,000	25,000	3.49	7,163	460	1.0	460	0.61	24,000	0.30
C-130	84,000	50,000	4.36	11,468	360	1.0	360	0.48	1,000,000	0.01
C-5A	190,000	25,000	1.62	15,432	380	1.0	380	0.51	400,000	0.04
P-3	68,000	50,000	3.45	14,493	400	1.0	400	0.53	240,000	0.06
F-14	30,000	100,000	8.58	11,655	285	1.0	285	0.38	UNLIMITED	-
									$\Sigma n/N =$	0.41

11.9.2 Asphalt Concrete Overlay Design Example. An asphalt concrete overlay is to be designed for the same runway and traffic conditions presented in paragraph a. above. Table 23 summarizes the calculations to determine the required single slab thickness for the primary traffic area. As summarized in the previous example, a single slab thickness of 13.5 inches (345 mm) is needed for the primary traffic areas. The "F" factor is determined to be 0.84 from figure 27 for an effective k value of 250 pci.

The condition factor Cb is determined to be 0.90 for a PCISTR of 70 from Figure 26.

The required asphalt concrete overlay thickness for channelized (primary) traffic areas is calculated as follows:

To = 2.5 (FTn - CbTe)

To = 2.5 (0.84*13.5 - 0.90*9.0)

To = 8.1 inches (rounded to 8.0 inches (203 mm)

The required asphalt concrete overlay thickness for unchannelized (secondary) traffic areas is calculated as above and is determined to be 8.1 inches (229 mm). This is rounded to 8.0 inches (203 mm).

SECTION 12: AIRCRAFT AND ENGINE RESTRAINT SYSTEM PROOF LOAD TESTING REQUIREMENTS

12.1 Purpose. This inspection and test is to determine that all component parts of land-based aircraft and engine restraint systems are in a safe and properly maintained operating condition.

12.2 Inspection and Test Frequency. The system consists of a restraint fitting (Type XIII, XI-A, etc.) fastened to a steel embedment which is anchored in the concrete foundation. Load test the system at the following intervals:

a) after initial installation. Load test new installations prior to operational use;
b) annually;
c) after repair and/or replacement of any of the components (i.e., restraint fittings, bolts, etc.);
d) due to visual signs of deterioration.

After completion of a proof test, NAVAIR requires a non-destructive inspection (NDI) of all restraint fittings and any operational hardware used during the test. When a restraint fitting is removed for NDI after a proof test and then re-installed, it is not necessary to re-test the fitting at that time. Refer to NAVAIR Technical Manual 17-1-537. Aircraft Securing and Handling Procedures with Organizational, Intermediate, and Depot Maintenance For Aircraft Restraining Devices and Related Components for information on NDI requirements.

12.3 Description of System. The proof load system (Figure 28) generally consists of: a hydraulic system for applying the load, which includes a hand pump and a single acting pull cylinder; special structural fixtures which interface with the proof load test fitting; and a chain assembly, turnbuckle and hardware to interface with the restraint fitting to be tested.

12.4 Proof Loads

a) Insure the proof load tests for Jet Engine Test Cells and Unabated Power Check Facility restraint systems are in accordance with Table 25. Use test procedures which gradually increase the load in increments as follows:

(1) Single XI-A ("+" configuration) - Proof-test load = 30,000 lbs.

(a) 10,000 lbs - hold for one minute
(b) 20,000 lbs - hold for one minute
(c) 25,000 lbs - hold for one minute
(d) 30,000 lbs - hold for ten minutes

MIL-HDBK-1021/4
Change 2, 31 August 1992

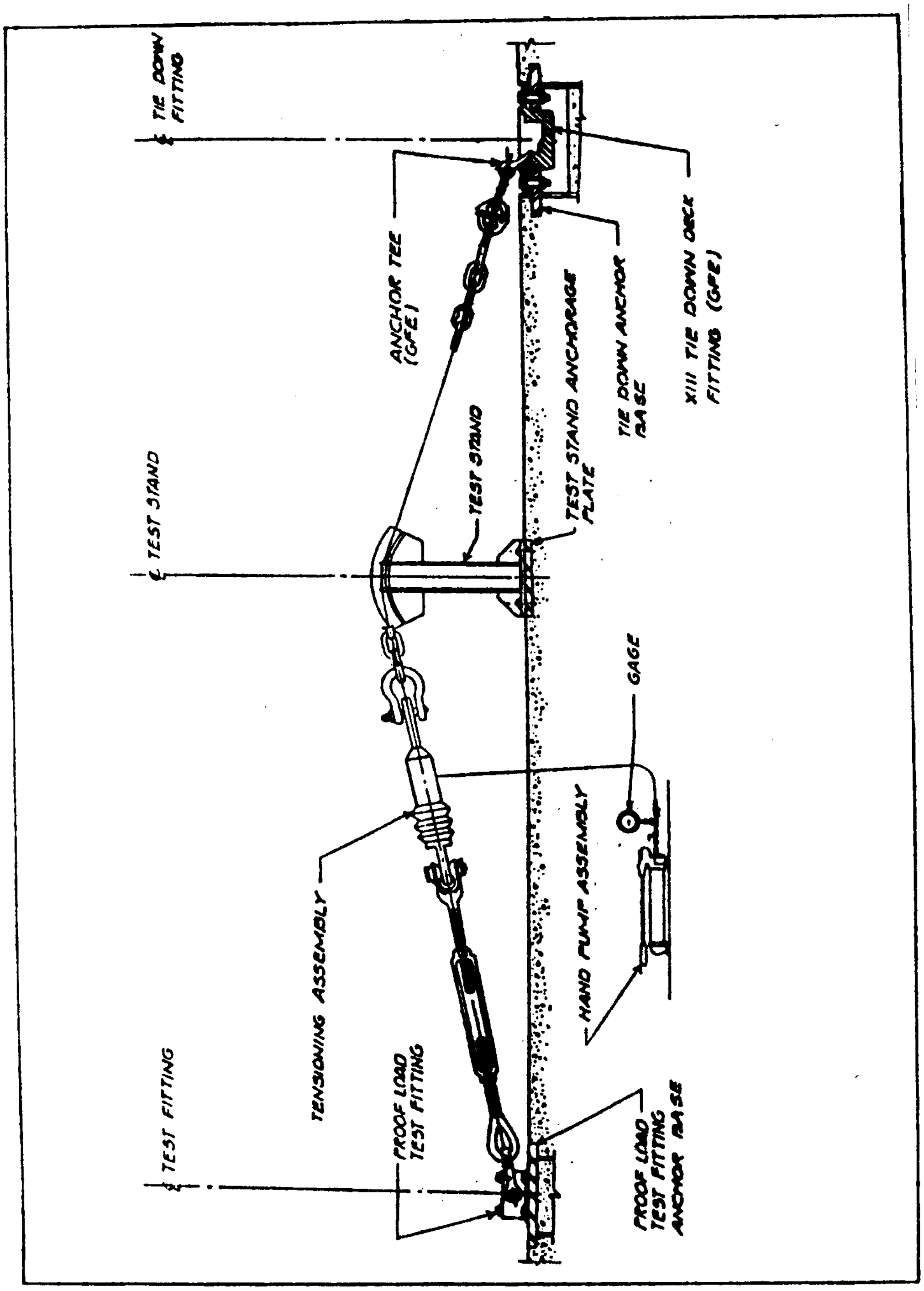


Figure 28
Proof Load Test System

(2) Single XI-A ("X" configuration) and Single F-2 - Proof-test load = 45,000 lbs.

(a) 10,000 lbs. - hold for one minute
(b) 30,000 lbs. - hold for one minute
(c) 40,000 lbs. - hold for one minute
(d) 45,000 lbs. - hold for ten minutes

(3) Dual XI-A ("X" or "+" configuration), Dual F2 and Dual XIII - Proof-test load = 60,000 lbs.

(a) 10,000 lbs. - hold for one minute
(b) 30,000 lbs. - hold for one minute
(c) 40,000 lbs. - hold for one minute
(d) 50,000 lbs. - hold for one minute
(e) 60,000 lbs. - hold for ten minutes

(4) Single XIII and Single F1-Proof-test load = 90,000 lbs.

(a) 20,000 lbs. - hold for one minute
(b) 45,000 lbs. - hold for one minute
(c) 70,000 lbs. - hold for one minute
(d) 90,000 lbs. - hold for ten minutes

b) Insure the proof test load for Aircraft Acoustical Enclosure (Hush House) restraint systems is in accordance with Table 26. Use test procedures which gradually increase the load in increments as follows:

(1) Single XIII, Single F1 and Hybrid - Proof-test load = 60,000 lbs.

(a) 10,000 lbs. - hold for one minute
(b) 30,000 lbs. - hold for one minute
(c) 40,000 lbs. - hold for one minute
(d) 50,000 lbs. - hold for one minute
(e) 60,000 lbs. - hold for ten minutes

12.5 Inspection and Test Procedures. Induce tension in the chain assembly by slowly pumping the hydraulic hand pump until the assembly is taunt. When the load is applied, keep personnel clear of the immediate test area.

Increase the load gradually in increments. After each load increment and when the final proof load is reached, the hydraulic safety valve should be closed and held as indicated above. The restraint system should be remotely observed during the loading phases. If any indication of cracking or deformation is apparent, the load should be gradually released and the test terminated.

After the loading phase is completed and the load released, the restraint system components (fittings, bolts, anchor base and concrete in the immediate area) should be inspected for signs of distress. Fittings and anchors showing such distress are unacceptable and shall be replaced.

12.6 Scheduling and Performance Responsibility. Scheduling of inspections and tests is the responsibility of the Commanding Officer of a Public Works Center or the activity Public Works Officer. Inspection and tests shall be made by qualified personnel on the cognizant activity's rolls, except:

a) Where inspection responsibility has been assigned to the Commanding Officer of a Public Works Center.

b) Where Commanding Officers of major or lead activities are responsible for performing the maintenance of Public Works and Public Utilities at adjacent activities.

c) Where it may be impracticable to employ qualified personnel for such inspections and tests because of the limited workload. In such situations, request assistance in obtaining inspection and test services from the appropriate Engineering Field Division Commander/Commanding Officer. The Engineering Field Division Commander/Commanding Officer shall arrange for the performance of these inspection and test services by an activity, having qualified personnel, located near the requesting activity, or by contract.

d) The static pull test indicated in paragraph 12.5 should be performed only under the guidance and supervision of an engineer.

e) Perform Structural Control Inspections by qualified activity personnel, or by contract.

12.7 Inspection and Test Results

12.7.1 Inspections and Tests by Activity. Promptly initiate necessary action to correct deficiencies found during inspections and tests.

12.7.2 Inspections and Tests by Engineering Field Division. This covers inspections and tests performed by EFD personnel or by contract.

a) Staff furnish the activity a list of deficiencies found and required corrective action.

b) Contract furnish the activity a copy of the contractor's deficiencies report.

12.7.3 Action on Deficiencies. If deficiencies found by inspections and tests are such that the Aircraft Power Check Facility is considered unsafe, the Facility should be secured until such time as the deficiencies are corrected.

12.8 Form. The Facility Condition Report (as detailed in MO-322, Inspection of Shore Facilities, Vol. I) may be used for recording deficiencies. It should be dated and signed by cognizant personnel responsible for conduct of the inspection and test. A copy of the Inspector's Report should be forwarded to the activity's Aircraft Intermediate Maintenance Department.

Table 25
Jet Engine Test Cells and Unabated Power Check Facility
Restraint System Proof Lands/Pull Angles

Deck Fitting(s) Type	Maximum Safe Working Load Pounds	In-Service Usage	Proof Test-Load Pounds	Proof-Test-Angle Degrees	NOTE
Single XI-A ("+") Configuration	15,000	In-Airframe Run-up-and/ or Out of Airframe Engine Run-up	30,000	15	(1)
Single XI-A ("X") Configuration	22,500	In-Airframe Run-up and/ or-Out of Airframe Engine Run-up	45,000	15	(2)
Dual XI-A ("X" or "+") Configuration	30,000	Out-of-Airframe Engine Run-up	60,000	30	(3)
Single F2	22,500	In-Airframe Run-up and/ or-Out of Airframe Engine Run-up	45,000	15	(2)
Dual F2	30,000	Out-of-Airframe Engine Run-up	60,000	30	(3)
Single XIII	45,000	In-Airframe Run-up and/ or-Out of Airframe Engine Run-up	90,000	15	(4)

Table 25 (Cont.)
Jet Engine Test Cells and Unabated Power Check Facility
Restraint System Proof Lands/Pull Angles

Deck Fitting(s) Type	Maximum Safe Working Load Pounds	In-Service Usage	Proof Test-Load Pounds	Proof-Test-Angle Degrees	NOTE
Dual XIII	30,000	Out-of-Airframe Engine Run-up	60,000	30	(3)
Single F1	45,000	In-Airframe Run-up	90,000	15	(4)

NOTES:

(1) A 30,000 pount proof-load-test of a single, "+" configured Type XI-A fitting certifies a site for a maximum safe working load of 15,000 pounds.

Simultaneous, dual-engine, maximum testing (military and/or afterburner) is prohibited for all aircraft. All aircraft and engine testing exceeding 15,000 pounds of thrust is also prohibited.

(2) A 45,000 proof-load-test of a single fitting certifies a site to test all aircraft (except F-14A+/D) and all engines (except F110-GE-400). This does not include simultaneous, dual engine afterburner testing of F-4, F-14A and F/A-18 aircraft.

(3) An 60,000 pound proof-load-test on a dual set of fittings certifies a site to test all engines.

(4) A 90,000 pound proof-load-test on a single fitting certifies a site to test all aircraft (excluding simultaneous, dual-engine afterburner testing of F-14A+/D aircraft) and all engines.

Table 26
Aircraft Acoustical Enclosure Restraint System
Proof Loads/Pull Angles

Deck Fitting(s) Type	Maximum Safe Working Load Pounds	In-Service Usage	Proof Test-Load Pounds	Proof-Test-Angle Degrees	NOTE
Single XIII	30,000	In-Airframe Run-up	60,000	15	(1)
Single F1	30,000	In-Airframe Run-up	60,000	15	(1)
Hybrid	30,000	In-Airframe Run-up	60,000	15	(1) & (2)

NOTES:

(1) Simultaneous, Dual-engine, afterburner testing is prohibited for all aircraft.

(2) Applies to non-standard fittings at NAS Patuxent River, MD. and NAS Jacksonville, FL.

BIBLIOGRAPHY

Airport Pavement Design and Evaluation, AC 150/5320-6C, Federal Aviation Administration, 800 Independence Avenue, S.W., Washington, DC 20590

Concrete, Mindness, S., and Young, F.J., Prentice Hall, Incorporated, Englewood Cliffs, NJ 07632, 1981

Design of Concrete Airport Pavement, Packard, R.G., Portland Cement Association, 5420 Old Orchard Road, Skokie, IL 60077

Foundation Engineering, Peck, R.B., Hanson, W.E., and Thornblum, T.H., John Wiley & Sons, Incorporated, New York, NY 10016, 1974

Longitudinal Joint Systems in Slip-Formed Rigid Pavement: Volume III-Users Manual, Technical Report, RRD-79-4-III, Federal Aviation Administration, 800 Independence Avenue, S.W., Washington, DC 20590

Manual of Concrete Practice, American Concrete Institute (ACI), P.O. Box 19150, Redford Station, Detroit, MI 48219

Principles of Pavement Design, Yoder, E.J., and Witczak, M.W., John Wiley & Sons, Incorporated, New York, NY 10016

REFERENCES

NOTE: THE FOLLOWING REFERENCED DOCUMENTS FORM A PART OF THIS HANDBOOK TO THE EXTENT SPECIFIED HEREIN. USERS OF THIS HANDBOOK SHOULD REFER TO THE LATEST REVISIONS OF CITED DOCUMENTS UNLESS OTHERWISE DIRECTED.

FEDERAL SPECIFICATIONS, MILITARY HANDBOOKS, AND NAVFAC GUIDE SPECIFICATIONS: Available from the Defense Printing Service, Standardization Document Order Desk, Building 4D, 700 Robbins Avenue, Philadelphia, PA 19111-5094

FEDERAL SPECIFICATIONS

SS-S-200E	Sealant, Joint, Two-Component, Jet-Blast Resistant, Cold-Applied, for Portland Cement Concrete Pavement
SS-S-1401C	Sealant, Joint, Non-Jet-Fuel-Resistant, Hot-Applied, for Portland Cement and Asphalt Pavement
SS-S-1614A	Sealant, Joint, Jet-Fuel-Resistant Hot-Applied for Portland Cement and Tar Concrete Pavement

MILITARY HANDBOOKS

MIL-HDBK-1001/3	Architectural Acoustics
MIL-HDBK-1021/1	Airfield Geometric Design
MIL-HDBK-1021/2	General Concepts for Pavement Design
MIL-HDBK-1023/1	Airfield Lighting
MIL-HDBK-1028/1	Aircraft Maintenance Facilities

NAVFAC GUIDE SPECIFICATIONS

NFGS-02232	Base Course for Rigid Paving and Subbase Course for Flexible Paving
NFGS-02511	Bituminous Hot Mix Pavement
NFGS-02520	Portland Cement Concrete Pavement for Roads and Airfields
NFGS-02522	Joints, Reinforcement and Mooring Eyes in Concrete Pavements
NFGS-02577	Pavement Markings (Airfields and Roads)

Naval Facilities Engineering Command, Design Manual (DM) and Maintenance and Operation Manual (MO). Copies of DM's and MO's may be obtained from the U.S. Naval Publication and Forms Center, 5801 Tabor Avenue, Philadelphia, PA 19120

DM-5.04	Pavements
DM-7.01	Soil Mechanics
DM-7.02	Foundations and Earth Structures
DM-22	Petroleum Fuel Facilities
MO-322	Inspection of Shore Facilities, Vol I

REFERENCES (Cont.)

OTHER GOVERNMENT PUBLICATIONS:

UNITED STATES BUREAU OF RECLAMATION, P.O. Box 25007, Denver, Co 80225-0007

Concrete Manual 1979

FEDERAL AVIATION ADMINISTRATION, 800 Independence Ave., S.W., Washington, DC 20590

Field Survey and Analysis of Aircraft Distribution of Airport Pavement, Hosang, V.A.,

U.S. ARMY ENGINEER WATERWAYS EXPERIMENTAL STATION, Soils and Pavements Laboratory, P.O. Box 631, Vicksburg, MS 39180-0631

Lateral Distribution of Aircraft Traffic, Brown, D.N., and Thompson, O.O., July 1973

DEPARTMENT OF THE AIR FORCE, Air Force Publication Distribution Center, Publication Distribution Officer, Bolling Air Force Base, Washington, DC 20332

AFM 88-6	Rigid Pavement for Airfield Other than Army

DEPARTMENT OF THE ARMY, U.S. Army Adjutant General Publications Center, 1655 Woodson Road, St. Louis, MO 63114

TM 5-822-4	Soil Stabilization for Pavements,

NAVAL AIR TECHNICAL SERVICES FACILITY, Commanding Officer, 700 Robbins Avenue, Philadelphia, PA 19111-5097

NAVAIR Technical Manual 17-1-537	Aircraft Securing and Handling Procedures with Organizational, Intermediate, and Depot Maintenance for Aircraft Restraining Devices and Related Components

NON-GOVERNMENT PUBLICATIONS:

American Concrete Institute (ACI), P.O. Box 19150, Redford Station, Detroit, MI 48219

211.1	Standard Practice for Selecting Proportions for Normal, Heavyweight and Mass Concrete
305R	Hot-Weather Concreting
306R	Cold-Weather Concreting
544.3R	Specifying, Mixing, Placing and Finishing Steel Fiber Reinforced Concrete

REFERENCES (Cont.)

American Society for Testing and Materials (ASTM), 1916 Race Street, Philadelphia, PA 19103

C29	Unit Weight and Voids in Aggregate, Test Method for
C31	Making and Curing Concrete Test Specimens in the Field, Method of
C33	Concrete Aggregates, Specification for
C39	Compressive Strength of Cylindrical Concrete Specimens
C40	Test Method for Organic Impurities in Fine Aggregate for Concrete
C70	Test Method for Surface Moisture in Fine Aggregate
C78	Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)
C85	Test Method of Cement Content of Hardened Portland Cement Concrete
C87	Effect of Organic Impurities in Fine Aggregate on Strength of Mortar
C88	Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate
C94	Ready-Mix Concrete, Specification for
C123	Test Method for Lightweight Pieces in Aggregate
C127	Specific Gravity and Absorption of Coarse Aggregate, Test for
C128	Specific Gravity and Absorption of Fine Aggregate, Test for
C131	Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact
C136	Sieve Analysis of Fine and Coarse Aggregates, Method for
C138	Unit Weight, Yield and Air Content (Gravimetric) of Concrete
C142	Test Methods for Clay Lumps and Friable Particles in Aggregates
C143	Slump of Portland Cement Concrete, Test Method for
C150	Portland Cement, Specification for
C156	Water Retention by Concrete Curing Materials, Test Method
C173	Air Content of Freshly Mixed Concrete by the Volumetric Method, Test Method for
C192	Method of Making and Curing Concrete Test Specimens in the Laboratory
C207	Hydrated Lime for Masonry Purposes, Specification for
C227	Potential Alkali Reactivity of Cement-Aggregate Combination
C231	Air Content for Freshly Mixed Concrete by the Pressure Method
C232	Test Method for Bleeding of Concrete

REFERENCES (Cont.)

D1196 Methods for Nonrepetitive Static Plate Load Tests of Soils and Flexible Pavement Components for Use in Evaluation and Design
D1452 Practice for Soil Investigation and Sampling by Auger Borings
D1556 Density of Soil in Place by the Sand-Cord Method
D1557 Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10 lb. Rammer
D1586 Penetration Test and Split-Band Sampling of Soils
D1587 Method for Thin-Walled Tube Sampling of Soils
D1883 Bearing Ratio of Laboratory-Compacted Soils
D2216 Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock and Soil-Aggregate
D2419 Sand Equivalent Value of Soils and Fine Aggregate
D2434 Test Method for Permeability of Granular Soils
D2487 Classification of Soils for Engineering Purposes
D2488 Practice for Description and Identification of Soils
D2628 Preformed Polychloroprene Elastomeric Joint Seals for Concrete Pavement
D2922 Density of Soil and Soil Aggregate in Place by Nuclear Methods
D2937 Test Method for Density of Soil in Place by the Drive-Cylinder Method
D2940 Specification for Graded Aggregate Material for Bases or Subbases for Highways or Airports
D3017 Moisture Content of Soil and Soil Aggregate in Place by Nuclear Methods
D4318 Liquid Limit, Plastic Limit and Plasticity Index of Soils

Cumulative Damage in Fatigue, Miner, M.A., Transaction of the American Society of Mechanical Engineers (Vol. 67, 1945), 345 East 47th Street, New York, NY 10017

New Formula for Stresses in Concrete Pavement of Airfields, Westerguard, H.M., Transactions of the American Society of Civil Engineers (Washington, SC, 1945), American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017

Portland Cement Association (PCA), 5420 Old Orchard Road, Skokie, IL 60077

Computer Program for Airfield Pavement Design, Packard, R.G., 1967
Design and Control of Concrete Mixtures, 12th Edition, October 1979

REFERENCES (Cont.)

C260	Air-Entraining Admixtures for Concrete, Specification for
C289	Potential Reactivity of Aggregates (Chemical Method)
C403	Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance
C418	Abrasion Resistance of Concrete by Sandblasting
C457	Recommended Practice for Microscopic Determination of Air-Void Content and Parameters of the Air-Void System
C469	Test Method for Static Modules of Elasticity and Poisson's Ratio
C494	Chemical Admixtures for Concrete
C496	Test Method for Splitting Tensile Strengths of Cylindrical Concrete Specimens
C512	Test Method for Creep of Concrete in Compression
C535	Resistance to Degradation of Large-Size Aggregate by Abrasion and Impact
C593	Specification for Fly Ash and Other Pozzolan for Use with Lime
C617	Method for Capping Cylindrical Concrete Specimen
C618	Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement
C642	Test Method for Specific Gravity, Absorption and Voids
C666	Resistance of Concrete to Rapid Freezing and Thawing
C672	Test Method for Sealing Resistance of Concrete Structure Exposed to Deicing Chemicals
C684	Method of Making, Accelerated Curing, and Testing of Concrete Compression Test Specimen
C702	Method for Reducing Field Samples of Aggregate to Testing Size
C779	Test Methods for Abrasion Resistance of Horizontal Concrete Surfaces
C823	Practice for Examination and Sampling of Hardened Concrete
C827	Early Volume Change of Cementitious Mixtures
D75	Sampling Aggregates, Practices for
D98	Calcium Chloride, Specification for
D420	Soil and Rock for Engineering Purposes, Recommended Practices for Investigation and Sampling
D421	Practice for Dry Preparation of Soil Samples for Particle-Size Analysis of Soils
D422	Particle-Size Analysis of Soils
D427	Test Method for Shrinkage Factors of Soils